

TANK CONSTRUCTION

ERNEST C. BECK

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TANK CONSTRUCTION

*Relating principally to the Design, Manufacture
and Erection of Tanks in Mild Steel*

BY

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PREFACE

BESIDES an endeavour to present information likely to be of use in the practical design and construction of tanks, one of the main objects of this volume is to draw attention to the many problems involved in tank construction. Properly regarded, these problems are so full of interesting possibilities that they become absolutely fascinating to the practical engineer; while their commercial importance is at once so real and so great that the field of study which they offer cannot prove aught but profitable.

Present-day requirements, developments and tendencies indicate unquestionably the need for a more soundly reasoned basis of design, and more efficient, rapid and economical methods of construction, than have been commonly employed in the past.

There seems to be an impression in the minds of some that our knowledge concerning the principles involved in the design and construction of tanks is more or less complete; that the only means available for reducing the costs of production is by cutting down the thicknesses of the sheeting or omitting essential parts of the construction; and that anything in the way of practical investigation and research would be mere waste of time and energy.

No impression could be more completely at variance with the facts. Instead of our knowledge with regard to the subject being complete, it is shown in the following pages that, in some of the most commonly employed methods of constructing tanks, we do not even know how the sheeting acts in resisting the pressures of the contained liquid; while it is usual to "design" the sheeting on a basis of assumptions which certainly and obviously cannot be realised in the finished structure. Instead of economy in manufacture being limited to senseless "cutting" of material, it will be seen by the reader that few branches of engineering can offer such wide and rich fields for the exercise of skill in securing true

economy through the more effective use and disposition of material. And instead of investigation being unnecessary, it is shown that there are several—and these but a few of the most obvious—points on which practical research is urgently needed.

So far as the author is aware, no book has been published previously dealing with the practical design and construction of tanks, and it is hoped that the method of treatment here adopted may prove acceptable, not only to those engaged in the actual design, fabrication and erection of tanks, but also to many concerned with structures of a similar nature. In addition, the book contains much that should prove of interest and assistance to students of structural work generally.

The aim throughout has been to make the treatment broadly suggestive, rather than particular or exhaustive, partly in the hope of stimulating interest in the subject, as well as showing the scope for individuality in providing for local exigencies and meeting special requirements.

Not only has every endeavour been made to describe clearly and faithfully the various methods of design, manufacture and erection in common use; but, in addition, it will be found that a considerable proportion of the book is devoted to suggestions for improving those methods, commercially and scientifically. The practical application of the various principles involved is illustrated by means of numerous examples, typical of ordinary practice, completely worked out.

Among the parts of the work which are believed to be original, attention is particularly invited to: (1) the discussion regarding Economy of Form, in Chapter II—especially with regard to the influence of floors and roofs, of costs differing materially from that of the walls, upon the economical proportions; and also with regard to the latitude available for departure from the proportions giving maximum economy in the area of sheeting; (2) the suggested method for staying the walls of rectangular tanks by means of horizontal rails, in Chapter IV; (3) the investigation concerning the action and design of curbs and rails, in Chapter V; (4) the treatment for trough-bottomed rectangular tanks, in Chapter VI; (5) the suggested methods for simplifying the design, manufacture and erection of the roofs, walls and floors of cylindrical tanks, in

Chapter VII; and (6) the treatment for dished bottoms of elevated cylindrical tanks, in Chapter VIII.

The bulk of the work given in the following pages was published in the form of Articles contributed by the author to the *Mechanical World* during the years 1916-1920; and the author is indebted to the editor of that journal for permission to republish the work in book form. All the work previously published in article form has, however, been carefully and thoroughly revised, while some new matter has been added.

With the object of focussing attention the more clearly upon the matter of actual tank construction, and also to prevent the book from becoming large and costly, no attempt has been made to treat in detail the steelwork or other construction for supporting elevated tanks. Some general considerations regarding such work are given, and particular mention is made of a few important points; but for treatment of the foundations, stanchions, beams and bracings in detail, the reader is referred to the author's book on *Structural Steelwork* (Longmans, Green & Co.).

The author hopes to find, in the near future, an opportunity for presenting a treatment of bunkers, bins and silos, on lines similar to those here adopted for tanks.

ERNEST G. BECK.

London,
October 1920.

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TANK CONSTRUCTION

CHAPTER I

MATERIALS, PERMISSIBLE STRESSES AND RIVETING

1. *Introduction.*—The storage of water and other liquids in tanks, below, at, or above ground level, provides some of the most interesting and important problems with which the practical engineer is called upon to deal. Circumstances and conditions vary widely with different types of requirements, and, indeed, often with individual cases of the same type, so that no rigid rules or methods, either for design or construction, can be of general applicability. Appropriate methods—or, at least, appropriate modifications of some standard method—should be used to provide for and satisfy the special circumstances and conditions of each type, and, where necessary, of each particular instance.

All methods must, of course, rest more or less upon a broad basis of accepted principles, as regards both theory and practice, if satisfactory results are to be obtained. The form and proportions of each tank should be in accordance with the established conclusions of mensuration, except in so far as modifications are demanded by local requirements and exigencies; the stability, strength, and stiffness of the structure as a whole, and also of each part and connection individually, must comply with the laws of mechanics, including adequate provision for corrosion, based upon the results of observation and experience; and the relative arrangement of the component parts needs to be such as will facilitate and ensure the production of an economical and efficient structure, by lending itself to simple and satisfactory methods of manufacture.

It is surprising to find that these considerations, though obviously correct and sound from the commercial, as well as from the scientific,

point of view, are to a large extent ignored—at least, in this country. Apparently, special requirements and local conditions are often regarded as the dominating factors in design, to which all other matters must be sacrificed, whereas the position they should occupy is that of particular features of the problem, to be always properly met and treated, but in no case permitted to assume paramount importance to the exclusion or undue detriment of efficiency and economy.

Sometimes, of course, the position allotted for the tank, owing either to lack of proper consideration in a new layout by one who will not be called upon to design the tank, or to insurmountable obstacles, is such as to render a good design very difficult, if not impossible; but even so, much may be done (in the way of limiting the effects of the initial handicap) with care and a clear understanding of the principles involved. Moreover, such cases are less numerous than is generally supposed, and might be fewer if the facts were considered; but even if efficiency and economy in the design of the tank proper had frequently to be sacrificed, there would still be no reason or excuse for abandoning them.

Were tank construction placed upon the definite basis which it deserves, all competent designers would agree as to the essentials of any proposed tank; after providing for the local conditions and particular requirements of the case (which are, of course, largely matters of fact, and not of opinion), legitimate differences in design would be limited to such directions as the respective equipment and resources of the shops and yards which the individual designers knew would carry out their work if accepted, and to the skill of each designer in minimising the unavoidable departure from the established principles of economy and efficiency caused by the special exigencies. A design which differed radically from that by which the most advantageous results could be obtained at the lowest cost would proclaim its producer either incompetent or lacking in proper facilities for satisfactory manufacture. Legitimate competition in prices would be limited to such matters as the ability and power to secure the best values, both in buying material and in methods of manufacture, the margin of profit required, and the cost of transport from the yard to the site, in addition to those resulting from the differences in design mentioned above. Clearly,

there could not, in such circumstances, be any great divergence in prices, while the quality of the tank would be high, and practically constant, for all.

How different would this be from (and how desirable in comparison with) the existing state of affairs, in which orders are too frequently given for tanks designed without regard for economy of form, the resulting extravagance being masked by the use of inferior materials and shoddy workmanship, to the detriment of conscientious and capable designers who would give greater value out of all proportion to the slight increase (often only apparent, and not real) in cost.

A tank nearly always requires special construction of some kind to support it, and the cost of the tank should be considered in conjunction with that of its supporting work. Tanks of uneconomical form may (and frequently do) involve extravagance in the methods of carrying them, and in this way, even though the cost of the tank itself be reduced by means of unwarrantable and dangerous expedients, the actual outlay in respect of the structure as a whole may be unnecessarily increased, in addition to probable waste of valuable space which a better design would have rendered available for profitable use.

In the following pages an attempt is made to indicate, in some detail, the principles which should underlie the design and construction of the various types of tanks in general use; and to show how those principles may be applied in practice. Regard is paid to the relative importance of the various principles in various circumstances, and to the methods by which local conditions and special requirements may be met and satisfied without undue sacrifice of economy and efficiency; while it is also shown that even accepted methods of design, generally regarded as of high standard, have so far failed to take sufficient account of some important considerations, and have ignored others, which might well prove very useful on occasion.

2. *Materials of Construction.*—Only the best materials of their respective kinds, and the most suitable for the purpose, should be used for the construction of tanks. With material of uniform and reliable quality, the necessary provisions for strength and stiffness, and allowances for corrosion and other effects, may be properly

made in such a manner that the margins are of (more or less) known extent, and may therefore be relied upon.

Most of the material used for the walls and floors of tanks is comparatively thin, and slight inequalities in thickness are unavoidable. A small decrease in thickness when the total nominal thickness is not great may involve a relatively large proportionate reduction in strength, even with good material. In low-grade material, not only are such inequalities in actual thickness greater, but there are also wider variations in the quality of the material, and thus it is impossible to rely, with justifiable confidence, upon the results of calculations as regards strength.

A further objection to the use of inferior materials lies in the fact that they are difficult to work. Joints and seams must be truly closed in tank construction, and this is impossible if the plates are not flat, or if holes cannot be properly formed owing to brittleness or irregularities in the metal. Where a satisfactory structure is required, therefore, it will probably be found cheaper, as well as better, to use standard materials. There are, of course, some who, having no reputation to preserve, care nothing for satisfactory results—indeed, the only result they regard as satisfactory is the due receipt of the contract price, highly inflated by imaginary “extras,”—and these, recognising that all (and more than) they save by purchasing inferior material would be lost if good workmanship were attempted, employ such other methods of manufacture as will, aided by plentiful red lead and jointing devices, enable the tank to pass muster if there be no intelligent examination or test—though mention of inspection and tests in a specification is usually sufficient to destroy their interest in the proposal.

A tank so constructed is certain to give trouble sooner or later; and once it starts, the trouble is likely to continue until the tank is scrapped. If those who made it be asked for an explanation, they reply in vague terms, darkly hinting at the existence of some mysterious and malignant power¹ which all their “skill and care” has sometimes (as in the case under question) been insufficient to circumvent—not because their efforts were relaxed or small, but because the evil one was unusually wily. With such we have no

¹ The author has actually heard such foolish excuses put forward, and not always without success.

concern, for the simple fact is that, apart from inevitable deterioration or the effects of misuse and neglect, a leaky tank bears witness to bad material or faulty workmanship, and to nothing else.

Some years ago, large numbers of tanks were built of cast-iron plates, flanged for jointing, and provided with stiffening webs where necessary. The skin of cast iron may render assistance in minimising corrosion, but the material is notoriously treacherous when subjected to bending actions, and this has undoubtedly been the cause of several failures. Moreover, although cast iron is cheaper than steel, it is doubtful whether a tank, of specified capacity, could be built for less in cast iron than in steel, for the jointing flanges, fillets and webs, involve a considerable increase in weight which is largely avoided in steel, while the meeting faces of all flanges on the cast-iron plates must be planed and fitted with accuracy and care if tight joints are to be possible, which would cost more than the riveting for a steel tank. It would be difficult, also, to fit stays in a cast-iron tank.

Probably some few tanks, of comparatively small sizes, are still made in cast iron, but for general use, steel construction, consisting of plates riveted together, and suitably stiffened and stayed to resist the internal pressure, is preferred, and to such an extent that no attention need be paid here to cast iron as a material for tank construction.

Recently several important tanks have been constructed of reinforced concrete, and there are undoubtedly circumstances in which this material seems to be peculiarly suitable. Owing to the thicknesses being unavoidably great as compared with those which would suffice in steel, the weight of the tank is somewhat seriously increased—with good design, the weight of a reinforced-concrete tank would be not less than twice that of a tank having the same internal dimensions built in steel, in spite of the fact that the weight of reinforced concrete is less than one-third that of steel,—but in some cases this increase in weight might be of little importance compared with the weight of the contained liquid.

The principles which underlie the design for a reinforced-concrete tank are, however, very similar to those discussed in the following pages; and as it is only in special circumstances, and under peculiar conditions that reinforced concrete is likely to be used in preference

to mild steel plated work for the construction of tanks, it has been thought well to omit discussion of such methods in detail from the present volume.

For steel tanks, all material should be of British Standard Specification—*i. e.*—

Mild-steel Plates.—Tensile breaking strength between the limits of 28 and 32 tons per sq. in., with an elongation of not less than 20 per cent. in a length of 8 in.

Steel Rivets.—Tensile breaking strength between the limits of 25 and 30 tons per sq. in., with an elongation of not less than 25 per cent. in a length equal to 8 times the diameter.

Wrought-iron Rivets.—To be made from bars of “Best Yorkshire Iron.” Tensile breaking strength between the limits of 21 and 25 tons per sq. in., with an elongation of not less than 29 per cent. in a length equal to 8 times the diameter,

Rolled sections (*i. e.* joists, channels, angles, etc.) should be of mild steel, to the same specification as for plates.

3. *Riveting Generally.*—There are two conditions to be satisfied in the riveted joints of steel tanks—viz. strength and tightness,—and both are equally important. Economy in holes and rivets must sometimes be sacrificed to obtain tightness, but a good designer will endeavour, by means of skilful arrangement, to minimise such unavoidable waste. For instance, it may happen that joints and seams, owing to the commercial limits of sizes in plates and bars, occur in positions where the effects of fluid pressure are small, and hence the provision of riveting necessary for mere strength would be insufficient to prevent leakage. It will presently be shown that by careful and intelligent designing, the occurrence of such joints and seams may be limited to cases in which they are really inevitable, and also that undue waste may be prevented.

Satisfactory riveted work is both costly and difficult to obtain in this country, and for this reason some manufacturers regard all other factors in the design of a tank as subservient to the minimising of riveting. It is, of course, sound economy to eliminate all unnecessary or avoidable riveting, provided that other, and more important, aspects of the design are not adversely affected thereby; but there are, as has already been pointed out, considerations

besides the first cost of the tank itself which may not always properly be ignored.

Plate thicknesses should not be less than $\frac{1}{4}$ in., nor more than 1 in., and no rivets should be used having diameters less than $\frac{1}{2}$ in. or more than $\frac{7}{8}$ in.

There is no simple and reliable rule expressing the most suitable diameter for the rivet in terms of the plate thickness; many of those so far proposed give absurd results for some thicknesses—for instance, $d = 1.2\sqrt{t}$ (perhaps the best-known one) is reasonable for thicknesses less than $\frac{5}{8}$ in., but suggests a rivet diameter of 1 in. for $\frac{3}{4}$ in. plates, and $1\frac{1}{4}$ in. diameter for 1 in. plates, which diameters are too large for practical use, requiring enormous power to close the rivets properly. The linear relation—

$$d = 0.4 + 0.5 t,$$

d being the rivet diameter and t the plate thickness, fits practical proportions, for this class of work, better than any other, but the large extent to which circumstances as regards strength and tightness of joints and seams vary, the small range of plate thicknesses and rivet diameters practicable, and the fact that rivet diameters must alter by eighths, or, at the least (and not too often), by sixteenths, of an inch, render any such rule of little use except as a rough guide for a first approximation from which modification may commence.

4. *Setting Out Rivet-holes.*—Rivet-holes, if they are to be formed by ordinary single punching methods, should be carefully set out by means of properly constructed wooden templets, which, for tank work, are usually very simple.

Multiple or rack methods may possess some advantages in quickness as compared with the single-formation methods, but they have disadvantages also in greater liability to breakage in the punches or drills, and the holding-up of the whole machine if one punch or drill breaks. Moreover, such methods are apt to involve limitations in the pitch and arrangement of rivets which cause loss of economy in the work as a whole.

An important point (which is surprisingly often lost sight of) is that *every rivet-hole must be filled with a rivet*; and, since the cost of filling a hole must be added to that of forming it, a cheap

method of forming holes, if allowed to form more holes than are necessary, may actually be the means of increasing the cost of the work as a whole.

The holes in the templets need not be the same size as those for the rivets which they locate; they are generally made to some convenient size (say, $\frac{3}{4}$ in. diameter), and kept uniform for all templets. If the rivet-holes are to be punched, the templet is laid on all the pieces to which it relates, one at a time, and the positions of the holes marked on the steel by means of a centre-punch, formed to fairly fit the holes in the templet, and having a central projection (or "nipple") with which the indentation is made. By this means, the markings on all pieces should be identical, no matter how many pieces there be.

If the holes are to be drilled, only one piece need be marked with the punch, and this piece may then be carefully drilled to form a "master" for all similar pieces. A number of pieces may then be clamped or otherwise fastened together, with the "master" piece on top, and the drill sent through all thicknesses at each mark in one operation.

5. *Punched Holes*.—If the holes are punched it is usual to employ a "nipple-punch"—i. e. a punch having a small conical projection at the centre of the circle formed by the cutting edge. The work is placed on the nest so that this projection (or "nipple") enters the indentation made by the centre-punch when marking-off, and thus it is ensured that the holes shall be formed truly in the positions marked.

It has long been the practice of engineers to specify that rivet-holes shall be drilled, but that if the manufacturer prefer to do so he may punch the holes slightly (usually $\frac{1}{8}$ in.) smaller than that necessary for the finished rivet, afterwards broaching or reaming to the final dimensions so that the material damaged by the punch shall be removed.

The latter method was, until recently, standard practice in the best yards; and is still very generally adopted, though drilled holes are becoming much more common than formerly. Even from the manufacturer's point of view, however, there has always been a drawback in connection with punched holes. This drawback consists in a lengthening or wrinkling of the pieces along the rivet

centre-lines, giving additional trouble in straightening and other adjustments necessary before the rivets can be driven.

6. *Effects of Punching.*—The material apparently shrinks from the punch every time a hole is formed, the consequence being that the bar is stretched along a row of holes. This lengthening often amounts to 0·1 per cent., or $\frac{1}{8}$ in. in a length of 10 ft.; it varies, of course, with the thickness of the metal, the form and section of the bar, and the size, pitch, and position of the holes.

Now, the first effect of this lengthening is different in different cases. If the bar be an ordinary flat, only a few inches in width, the length will simply increase; if it be a wider plate, with the holes close to the edges, the length will increase but slightly, if at all, because the main body of the plate resists stretching; but the strips along the lines of rivet-holes will stretch, and the result is a series of buckles or corrugations throughout the length. Such buckles are very stiff, and cannot be removed by bolting or riveting the pieces together; if the rivets be driven without first removing the corrugations, the majority of the rivets will be rendered loose or otherwise defective on cooling, and the seam cannot possibly be rendered tight against leakage. If one limb of an angle bar be punched the bar will curve, the punched limb increasing slightly in length, while if both limbs be punched the whole bar will stretch, and curve about the root, in addition to buckles being formed in the outer portions of the limbs.

The ultimate effect of all this is that the processes of marking, holing, assembling, and riveting do not follow in uninterrupted sequence. Nearly every piece must be straightened after punching as well as before marking, and, unless the length be small, further reaming is necessary when the work is assembled, before riveting can commence.

Consider, for instance, a row of rivets connecting the floor or side plates of a tank with the curb angle. The angle will stretch more than will the plate in punching, and it is not unusual for the holes at the ends to be 0·1 in. out of line when the work is assembled. The rivet would not pass through such an aperture, of course, so a reamer must be set to work. Now, there being only the two thicknesses, the reamer will cant in the hole unless special means be adopted to support it at both ends, and the hole, besides being

of irregular shape in both pieces (and therefore almost impossible to fill properly), will not be at right angles to the pieces. Even if the reamer be supported to keep the axis of the hole square with the surfaces of the pieces connected, the hole in the plate will be reamed on the opposite side from that on which the hole in the angle is reamed, making each hole oval in shape. Hence there would be two crescent-shaped spaces into which the material of the rivet must be forced if the hole is to be completely filled, which could seldom, if ever, be done. By commencing the riveting at the centre of the length, the amount by which the extreme holes are out of line may be minimised, but it cannot be eliminated by such means. The difficulty may sometimes be overcome by punching the holes smaller, and not reaming at all until the work is assembled; but the canting of the reamer when only two thicknesses are to be reamed, with the holes in each out of line, must be prevented if such means are to be effective.

If, owing to irregularity in shape, a hole be not completely filled by its rivet, some portions of the rivet will not be in contact with the metal in which the hole is formed, and hence no reliance can be placed upon the capacity of the rivet to transmit force from one piece to the other. In such cases, therefore, structural weakness may be inherent, as well as liability to leakage under the action of fluid pressure.

Punching has for many years been believed to seriously damage the metal around the hole, and this was possibly the reason for insisting upon such holes being broached or reamed. Recent observation, however, appears to indicate that, given a sharp punch and a well-fitting nest, the damage is less serious, both in nature and extent, than was formerly supposed, and there is a tendency to punch holes more nearly the finished size, leaving only a small amount for reaming. This might have the advantage of slightly reducing the cost of reaming, but the difficulties due to holes being out of line would certainly not be lessened.

7. *Drilled Holes.*—The introduction of high-speed tool steel for drills, and the improvements recently effected in electric driving for portable and other drilling machines, have done much to reduce the advantages in cost and time which punching formerly possessed over drilling for rivet-holes. In many of the best yards to-day, a

large proportion of the rivet-holes are drilled, and although it is probable that punching will always be useful in some circumstances (*e. g.* for a few holes of medium size, through small thicknesses, as gusset plates and the like), drilling is wisely being adopted to an increasing extent.

Less time is needed for marking and handling the work if the holes are to be drilled than if they are to be punched and reamed; while, since each hole is formed in all thicknesses at one operation from a single marking, no difficulties can arise through holes being out of line.

It is necessary to remove the burrs which the drill leaves on the surfaces as it emerges from each piece. This will be found requisite even though several pieces be drilled at one operation, each separate piece having a sharp rim around the hole, and unless these are all removed the pieces will not come together properly, nor will the rivets "cup-down" truly. Such burrs may be removed easily by running an old half-round file smartly along the row, knocking the burrs away. Similar burrs are, to some extent, formed also with punched-and-reamed holes.

8. *Nominal Rivet Diameters.*—Owing to the fact that the diameter of the hole must be larger than that of the rivet as obtained from the makers, to permit the insertion of the rivet at a temperature suitable for closing, there is considerable diversity of opinion and practice as to whether the nominal diameter refers to the rivet as purchased or the hole. Some hold that the indication of (for instance) a $\frac{3}{4}$ in.-diameter rivet on a drawing implies that the rivet-shank shall be $\frac{3}{4}$ in. diameter when cold, and the hole $\frac{1}{32}$ in. or $\frac{1}{16}$ in. larger; others work on the basis that the hole is $\frac{3}{4}$ in. diameter, and the rivet (as purchased) slightly less.

If the rivet, after driving, completely fill the hole, the latter method has the advantage in that the resistance of the rivet, and also the reduction in area of the pieces through which it passes (of importance when the pieces are in tension), may be properly calculated on the basis of a $\frac{3}{4}$ in.-diameter rivet and hole; whereas the former method would give a rivet resistance greater, and a tensile resistance (of the pieces connected) less, than those calculated for a rivet and hole both $\frac{3}{4}$ in. in diameter. On the other hand, if the finished rivet does not completely fill the hole, the former method

would appear to be preferable, since the rivet resistance would in most cases be lowered more by a reduction of $\frac{1}{16}$ in. in its diameter than would the tensile resistance of the pieces connected through an increase of $\frac{1}{16}$ in. in the diameter of the hole.

When each hole is drilled with all the pieces assembled (the drill being sent through all the pieces at one operation), and the rivets are properly driven by hydraulic pressure or pneumatic percussion machines, the holes are, in fact, completely filled, and hence, for such work, the nominal diameter may be the diameter of the hole.

Endeavours are being made to bring about the adoption of a standard for practice in this matter; and it appears probable that, at least for rivets driven by machine, the standard will be that the hole shall be of the stated diameter, and the rivet-shank as purchased only sufficiently less to permit of its insertion when hot.

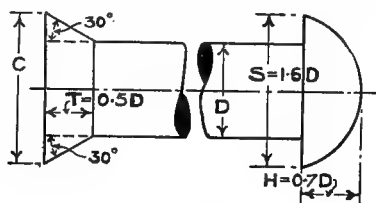


FIG. 1.

9. *Proportions of Rivets.*—Only snap and counter-sunk heads are now generally used for rivets in steel tank work. The proportions used by different makers vary slightly, but those given in Fig. 1 may be taken as representing good general practice. The dimensions of snap and countersunk rivet heads given in Table I. correspond to the proportions of Fig. 1.

TABLE I
(ALL DIMENSIONS IN INCHES)

Diameter of Rivet D.	Snap.		Countersunk.	
	H.	S.	T.	C.
$\frac{1}{8}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{8}$
$\frac{3}{16}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{3}{16}$	$\frac{1}{8}$
$\frac{1}{2}$	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{3}{8}$
$\frac{5}{8}$	$\frac{5}{8}$	$\frac{7}{8}$	$\frac{5}{8}$	$\frac{7}{8}$
$\frac{3}{4}$	$\frac{3}{4}$	1	$\frac{3}{4}$	1
$\frac{7}{8}$	$\frac{7}{8}$	$1\frac{1}{8}$	$\frac{7}{8}$	$1\frac{1}{8}$
1	1	$1\frac{1}{4}$	1	$1\frac{1}{4}$

Some rivets are made with a portion of the shank slightly tapered, as in Fig. 2. Immediately under the head these rivets are almost the full diameter, so that the rivet fills the hole tightly. Two advantages are secured by this method—viz. (1) there is less space into which the rivet must be driven to secure a properly filled hole, and (2) the rivet is kept central in the hole while being closed.

10. *Yard and Field Riveting.*—Practically all the yard riveting for tank work is done by hydraulic pressure or pneumatic percussion machines; only when no other means can be employed is hand-riveting used in the yard. The hydraulic pressure machine gives excellent results, and is much used for general steel-work; but it is less suitable for tank work because of the large distances to be spanned in many cases. In some yards where large tanks are made, yard riveting is limited to such items as fastening single

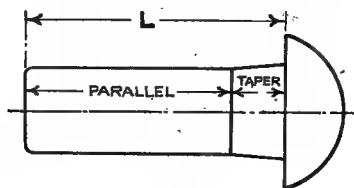


FIG. 2.

cover strips for butt joints to one plate, portions of framing which may conveniently be attached to the plates, and parts of the supporting or covering structural work; all other riveting being done at the site in course of erection. Such as these use hydraulic pressure machines for yard riveting, and either hand-riveting or pneumatic percussion machines at the site; but some prefer to use the latter method for all riveting, both in the yard and at the site.

For “field” riveting (*i. e.* that which must be done during erection and fixing at the site), either hand or pneumatic tools are employed, according to the magnitude and importance of the work. A small job would not bear the cost, in ordinary circumstances, of the plant necessary for pneumatic riveting; but with an undertaking of considerable dimensions the cost of installing such plant at the outset would usually be more than repaid by the reduction in the cost of riveting, the saving in time and trouble,

and the increase in the reliability of the work which would thus be obtained.

The range of temperature in which steel can be worked is narrow, and therefore, unless the rivet can be placed in the hole immediately after its removal from the furnace, the temperature may fall below the allowable minimum, with the result that the material will not submit to the riveter as it should, even though great power be employed. Particularly is this the case with small rivets, in which the initial amount of heat is inevitably small. For this reason, some engineers prefer to use wrought-iron rivets for all field riveting, since they can be worked over a greater range of temperature without injury. The majority, however, specify steel rivets throughout, and insist upon the necessary care being taken to obtain good field riveting.

Field rivets should never be more than $\frac{7}{8}$ in. in diameter, and whenever practicable they should be limited to $\frac{3}{4}$ in. diameter, owing to the difficulty of effectively working the relatively large amount of material by hand after the loss of heat during conveyance from the forge to the hole. This restriction is not so necessary in cases where pneumatic riveters are to be used at the site, and rapid transference of all rivets from the furnace to the hole can be ensured; but even there it is a wise precaution to allow for unforeseen contingencies.

11. *Faults in Riveting.*—If a rivet be burned or split, there will be little excuse for allowing it to pass. There are, however, other faults in riveted work which may escape notice in even the most rigorous examination, and which are almost impossible to remedy if discovered. Such should, therefore, be carefully guarded against.

One of these faults is the formation of a rivet head not co-axial with the shank. It is more likely to occur in hand or pneumatic percussion work than with rivets closed by hydraulic pressure machine, as the frame of the latter is too strong and stiff to permit such twisting of its jaws as would be necessary; it cannot happen with countersunk heads, of course, unless the rivet be too long. The most fruitful causes of this fault are: (1) Insufficient heat on the rivet; (2) excessive clearance in the hole—due either to small rivets or large holes; and (3) carelessness on the part of the workmen and their supervisors. In the first and second of these causes the

material prefers to bend over at the top rather than spread and flow, as it should, throughout its entire length. In the third cause it is less troublesome to simply turn the protruding point over than to drive the material carefully up into the clearance spaces, thus completely filling the hole first, and afterwards forming the snap so as to be truly concentric with the hole.

Apart from the unsightly appearance of work in which this fault exists, there is obviously an element of weakness, both in the rivets and in the whole seam or member in which it occurs. All specifications for high-class work contain a clause to the effect that pieces in which the rivet heads are not well and truly formed, co-axial with the hole, will be liable to rejection. It would be useless to suggest cutting out the defective rivets as a remedy, for it is often impossible to say which heads are faulty and which are not; the clause is therefore inserted as a lever, by means of which pressure may be brought to bear which will ensure the exercise of due care in this respect.

A point in connection with countersunk heads is worthy of notice. Some engineers insist on the surface being chipped level after riveting, while others prohibit such chipping on the ground that "it makes the rivets loose." Now, while it often happens that a countersunk rivet which appeared to be tight when driven is found to be slack after chipping, it does not follow that the chipping has caused the slackness; it is more likely to have simply revealed it. Instead of upsetting properly and filling the hole, the material sometimes (especially if the rivet be too long) spreads over the counter-sinking sufficiently to take all the pressure, and it is this rim which holds the rivet (apparently) tight. As soon as the projecting layer is chipped away, the slackness of the rivet is made known; and the worst of it is that, until it has been chipped, it is impossible to say whether such a rivet is tight or not.

12. *Lengths of Rivets for Ordering.*—The length of shank which should be allowed beyond the "grip"—*i. e.* the total thickness of the pieces to be connected—for filling the hole and forming the head depends upon the style of work (*i. e.* whether hand or machine), and also upon the rivet diameter and grip, since the hole space to be filled varies directly with the size and length of the hole. If the rivet be too short, there will not be sufficient material to

properly form the head after filling the hole; and if too long, a rim will be formed round the head, which may prevent the tight driving of the rivet, in a manner similar to that described above in connection with the countersunk head.

When ordering rivets from the manufacturers it is necessary to state the diameter, length under head to point, and type of head required. Particular care is necessary, in stating the diameter, to prevent the possibility of misunderstanding between actual and nominal diameters as explained above: if the holes are larger than the stated size, the rivet shank should be of the full stated diameter; and if the holes are the actual net diameter stated, the rivet shank should be some less diameter. In the former case a conspicuous note should be placed on the order to the effect that the rivets are to be of the actual diameters stated in the order, and in the latter case an equally conspicuous direction that the rivets are to be of diameters suitable for holes of the diameter stated. Perhaps the best method is to state the exact diameter of rivet shank required in every case for all orders, and the note may then be printed prominently on all order forms. The length to be stated is that represented as *L* in Fig. 2 (p. 13).

Lengths of rivets for ordering, for grips likely to occur in tank work, are given in Table II. These lengths have been found to give good results in practice, for hand and machine (hydraulic or pneumatic) riveted work.

Thus, a $\frac{3}{4}$ in.-diameter rivet to secure three $\frac{1}{2}$ in. plates, to be driven by machine, and to have snap head and point, would require to be $2\frac{7}{8}$ in. long under head to point. A rivet of the same diameter and grip, to be hand-driven, should be $\frac{1}{4}$ in. less in length—*i. e.* it should be ordered $2\frac{5}{8}$ in. long.

13. *Rivet Diameters.*—The determination of the diameter of the rivets to be used in a piece of work, if the best results are to be obtained, is not always so simple a matter as is sometimes supposed. There are, of course, first the questions of strength and tightness, and the necessary diameter of the rivets may be calculated by simple arithmetic after adopting, more or less arbitrarily, some particular disposal or arrangement of the rivets; but there are other considerations which should be taken into account before accepting the size so found. Regard should be paid to economy

TABLE II
LENGTHS OF RIVETS FOR ORDERING, IN INCHES

[illegible]

The above lengths are for machine riveting. For hand riveting the lengths should be reduced by $\frac{1}{4}$ in.

of labour, material, and weight; a proper relation should exist between the diameter of the rivet and the total thickness of the pieces through which it passes; and facility (and, therefore, economy

of labour) in driving the rivets should be secured. Each of these matters has a direct and important bearing on the proper diameter of the rivet to be used.

It has been found that a tight rivet and well-filled hole cannot be assured if the grip exceeds four times the diameter. A $\frac{5}{8}$ in.-diameter rivet should, therefore, not be used if the total thickness of the pieces through which it would pass exceeds $2\frac{1}{2}$ in.; the grip of a $\frac{3}{4}$ in.-diameter rivet should not be more than 3 in., and so on. Further, since the total cost of riveting is more nearly proportional to the number of rivets used than to their diameter, it is clearly more economical to use a small number of large rivets than a large number of small rivets, though this latter consideration may sometimes be outweighed by requirements for tightness against leakage, and other circumstances.

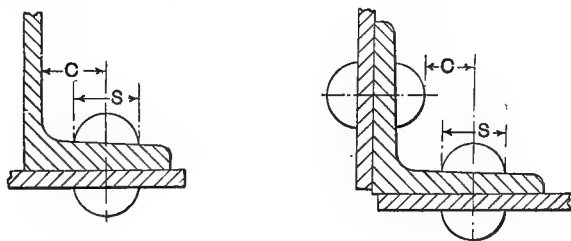


FIG. 3.

For facility in riveting it is necessary that adequate clearances be provided for the dies or tools, either hand or machine, both for closing and holding up the rivet. Two typical cases in which such clearances must be provided are indicated in Fig. 3. The distance C in each case should be not less than $\frac{1}{2}S + \frac{7}{16}$ in., S being as given in Table I. Difficulties in this direction may sometimes be lessened by judicious zig-zag spacing. It is better to allow for a slightly greater height of head than as given in Table I, as the rivets do not always close perfectly; the extra allowance should be $\frac{1}{16}$ in. for rivets up to $\frac{3}{4}$ in. diameter, and $\frac{1}{8}$ in. for the larger sizes. This determines the largest diameter of rivets which may be used with any angle, tee, channel, joist, or other rolled section, for any particular arrangement of pitch-lines.

The diameter may also be affected by limitations of pitch, etc.,

but the rivet finally selected should be of such diameter as will give the best agreement obtainable with all the foregoing requirements.

14. *Pitch and Arrangements of Rivets.*—In tank work the majority of joints and seams are single-riveted, and either lap-joints or butt-joints with double cover strips. Occasionally, where fluid pressures are great, double or treble riveted joints may be necessary, and the rivets are then “staggered”—*i. e.* arranged zig-zag—so that the sectional area of the pieces connected may be reduced as little as possible at any particular cross-section.

The pitch, for tightness of the joints against leakage, is almost invariably taken as three times the diameter of the rivets; but, while it should not be less than this, it may be slightly more where necessary, to keep a uniform pitch along a seam—*e. g.* when a pitch of three times the rivet diameter is not contained an integral number of times in the total length of the seam. No considerable increase of pitch should be permitted, however, unless the plates are stouter than is necessary for strength requirements, and will therefore work at a lower stress than would otherwise be allowed; and even then a pitch more than three-and-a-half or four times the diameter of the rivets is likely to cause trouble through “weeping” and the consequent corrosion.

Adequate provision must be made to prevent tearing of the plates beyond the rivets, and for this purpose the centre of a rivet should be at least one-and-a-half times the diameter of the rivet from the edge of any plate or bar through which it passes. Where the edge of the plate is not planed, and if the joint be important or heavily loaded, the distance between the centres of the rivets and the edges of the plates should be not less than twice the diameter of the rivets.

Other points regarding the most suitable arrangement for rivets, and the form of joint which should be adopted, in particular circumstances, will be indicated and discussed when considering the design of the various types of tanks.

15. *Rivet Resistances.*—The resistances of rivets to shearing and crushing (*i. e.* in bearing) are calculated on the nominal diameter, for the permissible stresses of 5·5 tons per sq. in. in shear, and 11 tons per sq. in. in bearing, except that the resistance of a rivet in double

shear is usually taken as 1.75 times the resistance of the same rivet in single shear. Experiments have indicated (and the more favourable loading of the rivet would lead to the assumption) that a rivet in double shear may carry twice the load which would be borne by the same rivet in single shear, but the Board of Trade, and other authorities, permit a load on a rivet in double shear of only 1.75 times the load allowed for a rivet of the same diameter in single shear—hence, all riveting in structures requiring the sanction or approval of such authorities must be designed accordingly. In work not requiring such approval, the resistance of a rivet in double shear is often taken as twice that of the same rivet in single shear, and it is probable that no great harm is thereby done.

Bearing resistances are calculated on the "projected area" of the actual bearing; thus, in a lap joint the bearing area would be taken as the projected area of the rivet in one plate thickness only.

Permissible shearing and bearing resistances of rivets, in single and double shear, and for various thicknesses of bearing, are given in Table III, double shear being taken as equivalent to 1.75 times single shear.

TABLE III
RESISTANCES OF MILD STEEL RIVETS

Di- ameter of Rivet in inches.	Cross- sectional Area in sq. in.	Shearing Re- sistances in tons, at 5.5 tons per sq. in.		Bearing Resistances in tons, at 11 tons per sq. in.											
				Thickness of actual bearing, in inches.											
		Single Shear.	Double Shear.	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
$\frac{1}{8}$	0.1963	1.08	1.89	1.38	1.72	2.06	2.41	2.75	3.09	3.44	—	—	—	—	—
$\frac{3}{16}$	0.3068	1.69	2.95	1.72	2.15	2.58	3.01	3.44	3.87	4.30	4.73	—	—	—	—
$\frac{1}{4}$	0.4418	2.43	4.25	2.06	2.58	3.09	3.61	4.13	4.64	5.16	5.67	6.19	—	—	—
$\frac{5}{16}$	0.6013	3.31	5.79	2.41	3.01	3.61	4.21	4.81	5.41	6.02	6.62	7.22	7.82	—	—
$\frac{3}{8}$	0.7854	4.32	7.56	2.75	3.44	4.13	4.81	5.50	6.19	6.88	7.56	8.25	8.94	9.63	—

Bearing resistances above the upper heavy stepped line are more than the resistances in double shear; hence, in these cases, shear is the determining factor. Bearing resistances between the heavy stepped lines are more than single shear and less than double shear; hence, the determining factor in these cases will be shearing for

single shear, and bearing for double shear. Bearing resistances below the lower heavy stepped line are less than the resistances in single shear; hence, in these cases, bearing is the determining factor.

It should be noticed that shearing resistances vary with the square of the rivet-diameter, the length being ignored, while bearing resistances vary as the product of the diameter and length of actual bearing; moreover, the stress allowed for bearing is twice that for shear. Hence, there may be considerable difference between the resistance of a particular rivet to shear, and the resistance of the same rivet to crushing, by reason of the relation borne by the plate-thickness to the rivet-diameter. Obviously, only the smaller of these two resistances may be taken as the resistance of the rivet, and loss of economy may arise in consequence. Where practicable, endeavours should be made so to design the riveting that all resistances shall be as nearly equal as may be, thus avoiding waste of material or labour.

For rapid checking, and in cases where an approximate estimate only is needed, it is convenient to memorise the shearing resistances of rivets as follows: $\frac{1}{2}$ in. diameter, 1 ton single shear, 2 tons double shear; $\frac{5}{8}$ in. diameter, 1.5 ton single, 3 tons double; $\frac{3}{4}$ in. diameter, 2.25 tons single, 4.5 tons double; $\frac{7}{8}$ in. diameter, 3 tons single, 6 tons double; and 1 in. diameter, 4 tons single, 8 tons double. These are round figures, easily remembered, and, as will be seen, not much in error. Bearing resistances may be easily calculated by the following simple relation—

$$R_b = \frac{(t_8 \times d_8)}{6},$$

where R_b is the resistance (bearing), in tons, and t_8 and d_8 are the plate-thickness (actual bearing) and rivet-diameter respectively, both expressed in eighths of an inch. Thus, for example, with a $\frac{3}{4}$ in.-diameter rivet bearing in a $\frac{5}{8}$ in. plate, t_8 and d_8 , expressed in eighths of an inch, would be 6 and 5 respectively, and hence the bearing resistance of the rivet would be: $R_b = \frac{(6 \times 5)}{6} = 5$ tons, which is very nearly correct, while the process is suitable for operation mentally. It is much easier to use than to describe.

In passing, it should be observed that the desirability of securing

(approximately) equal resistances to the various straining actions in a seam, constitutes another factor in the selection of the most suitable diameter for the rivets in any particular instance, to be taken into account simultaneously with the other considerations already mentioned in pp. 16-19.

16. *Friction in Riveted Joints.*—There is one fact concerning riveted joints which, though of real importance, is seldom mentioned—viz. that there is friction between the pieces through which the rivets pass. Having regard to the roughness of the surfaces of commercial steel plates and bars, and the considerable amount of tension set up in the rivets during cooling, there must be large frictional resistances to relative motion of the pieces, altogether apart from the rivet-resistances. Any proposal to estimate the magnitude of such frictional resistances would probably be regarded as unpractical, and calculations relating thereto would certainly be discounted by the opposing fact that both the roughness of the surfaces and the tension in the rivets are variable and practically indeterminate; but, on the other hand, it would be absurd to deny their existence.

Clearly, if it were possible to secure sufficient frictional effects by other means, rivets would become unnecessary, and this fact has recently been made use of in the construction of small vessels of a nature similar to tanks, when, owing to the difficulty and cost of obtaining good riveted work, the seams were welded by means of one of the new processes. In the ordinary way, of course, it is doubtful whether sufficient surface friction (*i. e.* between surfaces pressed together, but not welded) could be developed even for mere strength purposes, and it is probable that no seam could be made tight against fluid pressure by such means; but friction between the pieces connected must, beyond question, relieve the rivets to some extent in their resistance to shearing and crushing, and also tend to produce a more uniform distribution of stress over the pieces connected than would be the case were the rivets and plate-surfaces quite frictionless, as is assumed in calculations. At least it would appear that these facts might be taken into account where the calculated stresses in the rivets of a seam are slightly (say 4 or 5 per cent., as not infrequently happens) in excess of the agreed permissible stresses, especially if there be a good number of rivets

in the seam, for each rivet will assist in producing friction between the surfaces.

17. *Weight of Tankwork.*—The weight of tankwork is estimated on the basis that a cubic foot of steel weighs 489.6 lb. Other convenient figures, derived from this, are that a square foot of steel, one inch in thickness, weighs 40.8 lb., and a foot run of one inch square steel bar weighs 3.4 lb.

A cubic foot of wrought iron weighs 480 lb., so that a square foot of wrought iron, one inch in thickness, weighs 40 lb.

Cast iron weighs 454.5 lb. per cub. ft., and for weight calculations one cubic inch may be taken as weighing 0.263 lb.

All standard rolled steel sections have a definite weight per foot run, and joists, channels, angles, etc., should be ordered and calculated by their listed weights as well as by their over-all cross-sectional dimensions.

Approximate weights of rivets, as purchased from the manufacturers, are given in Table IV, and the various allowances at the foot of each column render the table applicable to rivets of any length, and with either snap or countersunk heads. It is useful for checking the number of rivets in a bag without counting, and also for estimates, etc., for the purposes of shipment and carriage.

In calculating the weight of riveted work it is only necessary to allow extra for the heads of the rivets, the shanks being accounted for by considering all plates, bars, etc., as solid. The usual practice is to count the heads, and multiply the number of them by the weight of one head, given at the foot of Table IV. It is necessary to note that each rivet has two heads.

Another method—which is, perhaps, slightly quicker, and which has the advantage of being independent of tables—is to count the number of heads, and consider each snap as the piece of shank from which it was formed; that is to say, take each head as a piece of round rod, of the same diameter as the rivet, and of length equal to one-and-a-half times its diameter. This is not strictly correct for rivets over $\frac{7}{8}$ in. diameter (being slightly excessive), but since such large rivets are seldom used, the rule may be followed with confidence for all ordinary work.

No allowance need be made for countersunk heads, of course.

The practice of estimating “by eye” the weight of rivet heads

TABLE IV—STEEL SNAP-HEADED RIVETS
WEIGHT IN POUNDS PER 100

Length under Head to Point in Inches.	Diameter in Inches.				
	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{3}{4}$	1
$1\frac{3}{8}$	11'8	—	—	—	—
$1\frac{1}{2}$	12'5	21'2	32'8	—	—
$1\frac{5}{8}$	13'2	22'3	34'4	50'0	69'5
$1\frac{7}{8}$	13'9	23'4	36'0	52'2	72'3
$1\frac{9}{8}$	14'6	24'5	37'6	54'3	75'1
2	15'3	25'6	39'1	56'4	77'9
$2\frac{1}{8}$	16'0	26'6	40'7	58'6	80'7
$2\frac{1}{4}$	16'7	27'7	42'2	60'7	83'4
$2\frac{3}{8}$	17'4	28'8	43'8	62'8	86'2
$2\frac{1}{2}$	18'1	29'9	45'4	64'9	89'0
$2\frac{5}{8}$	18'8	31'0	46'9	67'1	91'8
$2\frac{3}{4}$	19'5	32'0	48'5	69'2	94'6
$2\frac{7}{8}$	20'2	33'1	50'1	71'3	97'4
3	20'9	34'2	51'6	73'5	100'1
$3\frac{1}{8}$	21'6	35'3	53'2	75'6	102'9
$3\frac{1}{4}$	22'3	36'4	54'8	77'7	105'7
$3\frac{3}{8}$	—	37'5	56'3	79'8	108'5
$3\frac{1}{2}$	—	38'6	57'9	82'0	111'3
$3\frac{5}{8}$	—	39'7	59'5	84'1	114'0
$3\frac{3}{4}$	—	40'7	61'0	86'2	116'8
$3\frac{7}{8}$	—	41'8	62'6	88'4	119'6
4	—	42'9	64'2	90'5	122'4
$4\frac{1}{8}$	—	—	65'7	92'6	125'2
$4\frac{1}{4}$	—	—	67'3	94'7	127'9
$4\frac{3}{8}$	—	—	68'8	96'9	130'7
$4\frac{1}{2}$	—	—	70'4	99'0	133'5
$4\frac{5}{8}$	—	—	72'0	101'1	136'3
$4\frac{3}{4}$	—	—	—	103'3	139'1
$4\frac{7}{8}$	—	—	—	105'4	141'9
5	—	—	—	107'5	144'7
$5\frac{1}{8}$	—	—	—	109'7	147'4
$5\frac{1}{4}$	—	—	—	111'8	150'2
$5\frac{3}{8}$	—	—	—	—	153'0
$5\frac{1}{2}$	—	—	—	—	155'8
$5\frac{5}{8}$	—	—	—	—	158'5
$5\frac{3}{4}$	—	—	—	—	161'3
$5\frac{7}{8}$	—	—	—	—	164'1
6	—	—	—	—	166'9
$6\frac{1}{8}$	—	—	—	—	169'7
$6\frac{1}{4}$	—	—	—	—	172'5
$6\frac{3}{8}$	—	—	—	—	175'3
Weight of 100 snap heads . . .	4'2	8'2	14'1	22'4	33'4
Weight of 100 countersunk heads	3'5	5'4	9'4	15'9	22'3
Weight per inch of shank, per 100 .	5'6	8'7	12'5	17'0	22'3

in a piece of work, and expressing it as a percentage of the weight of the plates and bars making up the piece, is very convenient for those whose experience has been wide enough to enable them to judge what would be a fair allowance for any particular case; but as these allowances may vary between comparatively wide limits in different classes of work, considerable error may be caused by a slight lack of discernment. Unless the result is only required to be a rough approximation, it is always best to determine the actual number of heads, and calculate their weight by one of the methods explained above.

18.—*Permissible Stresses*.—With mild steel plates and bars of British Standard Specification it is usual to calculate plate-thicknesses upon the basis of permissible stresses not exceeding—

7·5 tons per sq. in.	in tension;
7·5 " "	in compression (pure, and in bending—not for stanchions and struts);
5·5 " "	in shear; and
11 " "	in crushing (or bearing).

For rivets, as has already been stated, the maximum permissible stresses are generally taken as 5·5 tons per sq. in. in shear, and 11 tons per sq. in. in bearing.

These represent good ordinary practice, and may be regarded as a basis from which modifications may start in exceptional circumstances, where either, on the one hand, a relaxation of the standard is reasonably permissible, or, on the other, greater stringency is required.

It is sometimes argued that, since the loading in tankwork is not subject to large or rapid fluctuation, higher stresses may be allowed, both in plates and rivets. Instances do occur, as suggested above, where conditions are very favourable, and advantage may (and should) then be taken of the circumstances to secure the utmost economy of material. Such instances are, however, exceptional, and should be so regarded. To design for stresses appreciably greater than those stated above, in ordinary circumstances, is unjustifiable and improper, as will be seen presently.

The maximum permissible stresses to be used as a basis for design depend upon several factors, mostly independent of each other. The more potent of these factors may be classified under five broad headings, thus: (1) The physical properties of the

materials; (2) the effects of processes of manufacture and handling upon the materials; (3) the nature and circumstances of loading; (4) the conditions under which the materials will work, and the permanence required; and (5) the so-called "factor of safety." As these matters, though of the greatest importance, are by no means well understood, we will consider them separately and in some detail, so that their influence upon practical tank structures may be properly appreciated. It is only by means of a clear understanding of the facts, and logical reasoning therefrom, that true economy can be obtained; and the flippant haziness with which the problems of the design and construction of tanks are so often treated results in much waste that could easily be avoided or counteracted.

(1) *Physical Properties of Materials.*—All structural design is based upon the assumption that the materials are elastic, and with mild steel this assumption is reasonably justified over a certain limited range of stresses. Beyond that range, however, the material becomes, to a considerable extent, plastic, and no basis for design under such conditions is practicable. For the purpose of designing commercial structures, the stress at which the material ceases to be elastic should be regarded as the ultimate strength, and in mild steel this stress is about one-half of that under which the material would actually break in tension.

If a part of a structure be strained beyond the elastic limit—*i. e.* beyond its power of recovery on removal of the load,—many things may happen, but it is not necessary either to discuss or to enumerate them here; nor is it necessary to differentiate precisely between "elastic limit" and "yield point." It is sufficient to notice that in a tank something much less spectacular than the actual breaking of a plate or piece may be sufficient to cause serious damage; with plates or rivets which have been stressed to such an extent that they are no longer elastic, it is evident that the tightness of the seams cannot be assured, and when that stage is reached the utility of a tank has practically been destroyed.

Moreover, ordinary calculations take account of primary stresses only, and assume either uniform distribution or uniform variation of stress over the whole section. There are, as a fact, always secondary stresses, some of which would be almost impossible to

estimate, but which are known to be highly important. By taking the permissible working stresses as stated above, therefore, and ignoring the secondary stresses, we are really only allowing a similar amount for the sum of all the secondary stresses—due to inequalities in the materials, and other defects, which are not taken into account in the calculations, but which exist none the less for that.

(2) *Effects of Manufacture.*—It is well known that commercial rolled steel bars are not perfectly straight, nor of constant cross-section, nor can they be made so. Steel plates, again, as bought from the mills, are neither flat nor of uniform thickness throughout.

During transport, pieces often become so further bent that they have to be again straightened before the processes of manufacture can proceed, and since such straightening is nearly always done without heating the material, parts must inevitably be strained beyond the elastic limit. Among the operations which are

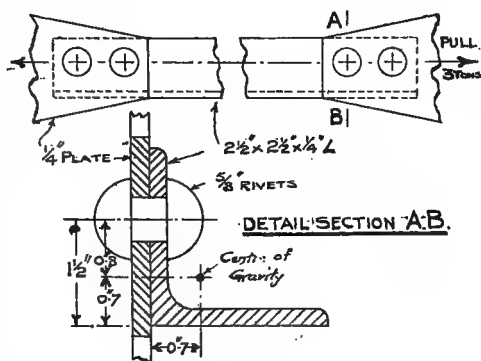


FIG. 4.

performed during manufacture, several cannot fail to set up initial stresses in the pieces connected, and the final result is that a member which has a calculated primary stress of 7.5 tons per sq. in. may be really subjected, at least in parts, to considerably more.

With some sections—angles and channels in particular—rivets cannot be driven so that their axes will lie in a plane which contains the centre-of-gravity line of the section, and consequently forces applied to a bar through such rivets are unavoidably eccentric, causing bending actions. As an example of what may occur, consider the case illustrated in Fig. 4, the angle bar acting as a tie. The primary stress, on the assumption that the direct tensile load is distributed uniformly over the net section, is about 3 tons per sq. in.; but there is also a bending moment of somewhere about 2 in.-tons with regard to both axes, and since the section modulus

about those axes is about 0.38 (inch units), the stress due to the bending action, to a first approximation, will be about 5 or 6 tons per sq. in. In addition there may be stresses due to the bending action caused by the weight of the bar if its axis be not vertical.

Stiffness and friction in the joints unquestionably reduce these stresses, but cannot eliminate them, and there are also other stresses due to irregularities in shape and inequalities in the material, which can only be guessed in any particular case.

In riveted seams, with the lap joint and single-covered butt joint connecting two pieces in tension, there is the well-known tendency to distortion as indicated in Fig. 5, and this must set up considerable additional stresses in the plates as well as in the rivets.

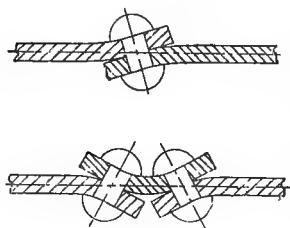


FIG. 5.

These and many other effects which cannot be prevented or calculated accurately, must be to a reasonable extent provided for. It has been estimated that in some classes of structural steelwork the secondary stresses together are not less than the primary stresses, and while they are probably

not so severe as that in tank work, there can be no doubt that they form a considerable part of the total stresses.

(3) *Conditions of Loading.*—Alternating and fluctuating loads cause more stresses than do steady loads of equal magnitudes. The loads in tank work are usually steady, but this fact cannot of itself be a sufficient reason for allowing higher stresses, because it is generally agreed that when varying loads have to be provided for, the permissible stresses should be reduced according to the range of variation.

There are instances, as will be seen later, where advantage may be taken of the circumstances of the stresses in adjacent material, to avoid using a thicker plate when the calculated stress is in excess of the maximum permissible by only a small percentage, but otherwise there is no justification in the steadiness of loading for raising the limits of working stresses.

In one way, the parts of a tank are less favourably placed than those of some other structures. In the latter it often happens that

a member which receives more than its due load can, by distorting slightly, obtain assistance from other members or construction. No relief can be found in a tank, however, except by reducing the pressure of the contained liquid, and this is usually prevented by some device for maintaining the level as nearly constant as may be.

(4) *Conditions of Working and Permanence.*—With a tank containing water which is not very frequently changed, there is usually little or no corrosion, and if the exterior surfaces be ordinarily well protected, the tank may be designed for the working stresses stated above. When a tank is liable to be emptied and filled in rapid and continual sequence, some allowance should be made for corrosion, which will then be more effective than with water which is merely stored.

It is usual to provide such allowance for corrosion by lowering the maximum permissible stress to 7 or 6.5 tons per sq. in., which, of course, has the effect of slightly increasing the thickness or other dimensions of the sections. It is difficult to justify this means, for, clearly, there can be no connection in fact between the working stress and the provision necessary on account of corrosion, and no distinction is made by it between pieces which have only one surface and those which have more exposed to corrosive action. Moreover, it does not provide equally on all sections, though the action of corrosion cannot so vary from point to point.

A more logical method is to design for thickness on the above basis of working stresses, and then to increase the thickness so obtained by some definite amount, say, $\frac{1}{8}$ in. or $\frac{1}{16}$ in., as experience indicates. If a piece were, by some means, found to be stressed (by primary calculation) below the maximum permissible, the net thickness required might be determined, and if the actual thickness provided a sufficient margin for corrosion nothing further need be added.

As is well known, corrosion due to water is more active in parts alternately wet and dry than in parts wholly wet or dry, and hence care should be taken to provide adequately over the range between high and low water levels.

If the tank is to contain some liquid which attacks steel with chemical action, data as to the rate of decomposition will be necessary, and provision should be made by additional thickness calcu-

lated according to the length of time the tank is required to last. In such cases, of course, frequent examination is necessary, so that local inroads—about cracks in seams, rivet heads, and similar places—may be detected and arrested.

Some tanks are continually subjected to the action of fumes—frequently of sulphur, as at railway depots; and here, again, the allowance for corrosion should be based on experience as to the rate of attack, and the degree of permanence required.

All such allowances should be liberal rather than the reverse, and based upon some reasonable estimate.

(5) *Factor of Safety*.—This is more properly a “margin for contingencies.” As a similar instance, on a railway platform one might say that, for safety when an express train is passing or approaching, one should not be less than 7 ft. from the edge. This would allow a margin of 7 ft. to provide for suction, possible projections from the train, involuntary movement, and such things, which experience has shown should be provided for. A platform 14 ft. in width might then be said to give a “factor of safety” of 2, but the essential fact is that a margin of 7 ft. is required, irrespective of the platform width beyond that margin.

If it were known positively that the train would pass at a low speed, that there would be no projections from it, and that no involuntary movement of any magnitude could occur, a distance less than 7 ft. could safely be allowed. In the case of mild-steel structures, the working stresses stated above leave a margin of about the same magnitude for secondary stresses, inequalities in the material, etc., and usually this is found to give satisfactory results. Sometimes (as when secondary stresses are known to be highly effective) a wider margin should be allowed; and sometimes a less margin will be sufficient.

One frequently hears arguments in favour of high working stresses based upon the shortness of life intended for the structure. Of course, if it could be ensured that the structure would be demolished at the end of a certain period, allowances for corrosion and similar effects might reasonably be made for that period only, but the intensity of pressure due to a certain head of water is not less if exerted for one year than if exerted for twenty years, and any considerable increase of permissible stresses on this account is

unjustifiable. The utmost that might be allowed would be to so design the parts that, by the end of the term stated, the total stresses from all causes would be in the neighbourhood of (but not likely to exceed) the elastic limit. Even this, however, is very seldom warrantable.

Moreover, structures (and tanks particularly) which are regarded as temporary when erected, have a knack of becoming very soon afterwards regarded as permanent, and it is difficult, if not impossible, to form a reliable estimate of the period during which they are likely to be retained.

The only circumstances in which the maximum permissible stresses stated above may be increased, are such as permit a reasonably exact computation of all the stresses,—secondary as well as primary,—or the strong probability that they will not exceed a certain amount. Then the margin between the total of all the stresses and that stress which marks the limit of elasticity might be only two or three tons per sq. in. Such cases seldom arise, however, and it is generally better to follow the course which experience has shown to be satisfactory, adopting the limits of working stresses given above, and, as a rule, ignoring secondary stresses.

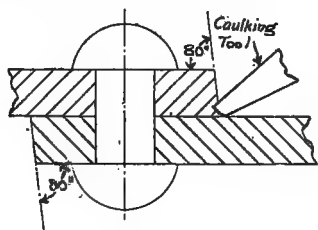


FIG. 6.

19.—*Caulking*.—No jointing material should be permitted in the riveted seams of tanks. All the necessary tightness against leakage may be obtained by caulking the plate edges, as indicated in Fig. 6. Caulking may be done by hand or by pneumatic machine, but the latter is rather apt to cause damage to the plates, owing to the difficulty of controlling the tool while being operated at a high speed. The tool should be rounded on the edge which bears upon the lower plate, as shown, to avoid cutting into that plate. Some authorities state that the caulked edges of the plates should be planed with a bevel, the angle being about 80° , as indicated in Fig. 6; but this is seldom done. Many caulk only the inner edges of the seams, where exposed to the contained liquid, leaving those outside the tank as riveted. This should be done with sufficient

thoroughness to prevent leakage, of course, for any liquid finding its way into the joint will probably cause trouble; but it is better to caulk all edges, so that moisture from the atmosphere and weather may not enter and set up corrosion where it would be difficult to see or check.

Butt joints have more edges to caulk than lap joints, but, being stiffer, the caulking is both less troublesome to do and more likely to be effective.

CHAPTER II

ECONOMY OF FORM

20.—*Economy of Form.*—The obvious and primary measure of the utility of a tank is its capacity for storage, while its cost, as a tank, is the cost of the shell. In many cases, of course, the tank must be carried upon some sub-structure (elevation being necessary, as well as capacity), and the cost of the sub-structure may be more than that of the tank itself, while it is often advisable to sacrifice economy in the tank so that a higher degree of economy may be obtained in the structure as a whole. These points will be illustrated later, but for the moment it is well to consider the tank by itself.

Now it might appear (and, indeed, has sometimes been stated) that the most economical form for a tank must be that which gives the greatest cubical contents for the least area of containing shell. This, however, ignores the very important fact that the cost is not always directly proportional to the area of the plating. The saving in area may be counterbalanced (or, at least, partly so) by a greater thickness of plate being rendered necessary in consequence, or the need for auxiliary stiffening, owing to the increased pressures of the contained liquid brought about by the particular form of the tank. Moreover, even with the same thickness of plate, the cost of a small area in a class of work involving much labour may be more than that of a larger area in simple work.

Thus, it will be seen that to claim that one form is more economical than another solely because the one requires a little less weight of material than the other, may be unjustifiable unless saving in weight be more important than any other consideration. It does happen, sometimes, that, by reason of the special circumstances and conditions, a saving in one particular respect is more desirable than a greater saving either in other respects or in general, but as

a rule, the economy to be sought is that which includes all aspects and secures the best return for the least outlay.

The foregoing remarks are not intended to imply that the form which gives the minimum area of sheeting for a specified volume is of no practical use, and may therefore be ignored; on the contrary, unless the particular circumstances of the case require that a certain form shall be used, that which gives the least area of sheeting should be determined, so that it may be seen whether it will give, or may be modified to give, the most economical design.

In comparing the suitability and efficiency, for tank work, of flat plates with those of plates bent to some curved shape, it should be noted that a flat plate must act as a beam, and hence, that only the equivalent of one-half of its cross-sectional area (one quarter in tension and one quarter in compression) can be fully stressed without exceeding the maximum permissible stress at the extreme fibre, whereas a curved plate, if bent to the proper shape, may have its whole cross-sectional area fully stressed in tension. Hence, although the cost of bending the plates is additional, the total cost may be reduced through the bent shape rendering permissible a less thickness of material than would be necessary with a flat plate.

The bending of plates to form parts of cylindrical and other similar surfaces is easily and cheaply done by cold-rolling, but for spherical and such shapes the bending is more difficult, and, consequently, more expensive; even these latter forms are useful in some cases, however, as will be seen presently.

A spherical tank would give the greatest possible contents for the least area of shell, but would have the disadvantage of being closed; it would also be difficult to support, and the work would be of a very costly nature. A hemisphere would be free from the first disadvantage, but not from the other two. There are other objections to such a form, which will present themselves on consideration, and all these together account for the fact that tanks of spherical shape are very seldom used.

With elevated cylindrical tanks, however, advantage may sometimes be gained by giving the tank a hemi-spherical dished bottom, so increasing the contents and saving the costly flat plate bottom and its supporting beams and joists. There are, of course, disadvantages as well as advantages in this method, and the total saving

in outlay may be comparatively small, but it is worth considering. Sometimes the bottom is made conical, but the best shape would, clearly, be in the nature of a semi-prolate spheroid, giving no bending actions in the plates. This matter is more fully discussed in pages 52-57, 67-69, and in Chapter VIII, dealing with the tanks in which it is of use and importance.

The next most economical form is the cylindrical, and many tanks, especially of large capacity, are made in this form. For a variety of purposes, however, the rectangular shape is both suitable and widely used, and we will consider it first as regards economical proportions.

21.—*Tanks Square on Plan.*—Since a square is the greatest rectangle which a given perimeter can contain, it seems reasonable to assume that, for economy of sheeting area, one face at least should be square. Let the tank be square on plan, both its sides being b in length, and the depth d . Then the volume will be—

$$V = b^2 d, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and the area of the shell—

$$A = 4 b d + b^2, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

supposing that there is no top or lid.

From (1), $d = \frac{V}{b^2}$, and, inserting this value for d in (2)—

$$A = \frac{4 b V}{b^2} + b^2 = \frac{4 V}{b} + b^2.$$

Let the volume be assumed constant, and differentiate A with respect to b , to find the value of b which gives the least for A . Then—

$$\frac{d A}{d b} = - \frac{4 V}{b^2} + 2 b,$$

and for minimum A , the expression must be zero. Hence—

$$\begin{aligned} 2 b^3 &= 4 V. \\ \therefore b^3 &= 2 V = 2 b^2 d, \end{aligned}$$

whence

$$b = 2 d. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If the top or lid be included—regarding the roof as equivalent in cost, per unit of area, to the sheeting—equation (2) becomes—

$$A = 4 b d + 2 b^2, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

the volume remaining unaltered, which gives as the most economical value of b —

$$b = d \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

—i. e. a cubical form.

Considerable departure from these proportions may be made, however, without much loss of economy (of area only, it must be remembered), as will be seen from the following numerical example.

Take $b = 10$ ft., and $d = 5$ ft. Then, from (1) and (2)—

$$V = 500 \text{ cub. ft., and } A = 300 \text{ sq. ft.}$$

Now let $d = 4$ ft., V being constant. Then $b^2 = \frac{500}{4}$ sq. ft.

$$b = \sqrt{125} = 11.18 \text{ ft.}$$

$$\begin{aligned} \therefore A &= 4(4 \times 11.18) + 11.18^2 \\ &= 178.88 + 125 = 303.88 \text{ sq. ft.} \end{aligned}$$

If $d = 3$ ft., $b^2 = 166.67$ sq. ft., $b = 12.91$ ft.—

$$A = 154.92 + 166.67 = 321.59 \text{ sq. ft.}$$

If $d = 6$ ft., $b^2 = 83.33$ sq. ft., $b = 9.13$ ft.—

$$A = 219.07 + 83.33 = 302.4 \text{ sq. ft.}$$

If $d = 7$ ft., $b^2 = 71.43$ sq. ft., $b = 8.45$ ft.—

$$A = 236.7 + 71.4 = 308.1 \text{ sq. ft.}$$

The difference is more marked as the tank becomes shallower, but is small for a considerable range of variation, and advantage may be taken of this fact to save labour in manufacture.

The conditions of this case are as indicated in Fig. 7, the variations in proportions consequent upon an increase or decrease in the depth (from the relation $b = 2d$ shown in equation (3) to give minimum area of shell) being shown by dotted and chain-dotted lines respectively.

We will now consider the case of a similar tank, but having a roof; and we will deal with it first on the basis of an assumption that the cost of the roof per square foot will be equal to that of the tank shell proper.

For such conditions, equation (5) shows the cubic form to be the most economical as regards area of sheeting. To facilitate comparison, let us consider a tank having a capacity of 500 cub. ft., as in the example relating to a tank not covered at the top.

The cubic form will give length, breadth, and depth equal to—

$$d = \sqrt[3]{500} = 7.94 \text{ ft.},$$

whence the area of the shell surface will be—

$$A = 6 d^2 = 6 \times 63 = 378 \text{ sq. ft.},$$

of which 63 sq. ft. will be in the roof, and 315 sq. ft. in the shell proper.

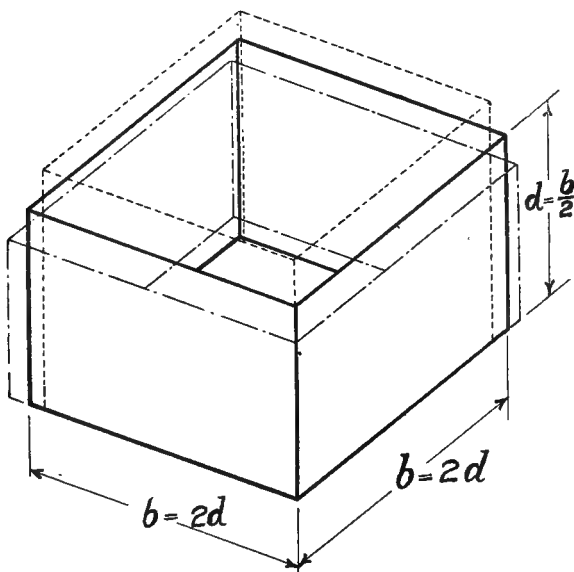


FIG. 7.

Keeping the plan square and the volume constant, the effects of making the tank shallower will be as follows—

$$\text{If } d = 7 \text{ ft., breadth (or length)} = b = \sqrt{\frac{500}{7}} = \sqrt{71.4} = 8.45 \text{ ft.}$$

$$\begin{aligned} A &= 4 b d + 2 b^2 = 2 b (2 d + b) = 16.9 (14 + 8.45) \\ &= 16.9 \times 22.45 = 379 \text{ sq. ft.}, \end{aligned}$$

of which 71.4 sq. ft. will be in the roof, and 307.6 sq. ft. in the shell proper.

$$\text{If } d = 6 \text{ ft., } b = \sqrt{\frac{500}{6}} = \sqrt{83\frac{1}{3}} = 9\cdot13 \text{ ft.—}$$

$A = 18\cdot26 (12 + 9\cdot13) = 18\cdot26 \times 21\cdot13 = 386 \text{ sq. ft., of which } 83\frac{1}{3} \text{ sq. ft. will be in the roof, and } 302\cdot7 \text{ sq. ft. in the shell proper.}$

The effects of increasing the depth will be as follows—

$$\text{If } d = 9 \text{ ft., } b = \sqrt{\frac{500}{9}} = \sqrt{55\frac{5}{9}} = 7\cdot45 \text{ sq. ft.—}$$

$$A = 14\cdot9 (18 + 7\cdot45) = 14\cdot9 \times 25\cdot45 = 379\cdot2 \text{ sq. ft.,}$$

of which $55\cdot55 \text{ sq. ft. will be in the roof, and } 323\cdot65 \text{ sq. ft. in the shell proper.}$

$$\text{If } d = 10 \text{ ft., } b = \sqrt{\frac{500}{10}} = \sqrt{50} = 7\cdot07 \text{ ft.—}$$

$$A = 14\cdot14 (20 + 7\cdot07) = 14\cdot14 \times 27\cdot07 = 382\cdot8 \text{ sq. ft.,}$$

of which $50 \text{ sq. ft. will be in the roof, and } 332\cdot8 \text{ sq. ft. in the shell proper.}$

This case is illustrated in Fig. 8, the dotted lines showing the changes in the proportions due to increase or decrease in depth with constant volume and keeping the plan square.

From the calculations it will be seen that as the tank is made shallower, the sheeting proper (*i. e.* in the walls and floor—the actual containing surfaces) becomes more efficient, approaching maximum economy as the proportions become more nearly the ideal $b = 2 d$ of equation (3). Reduction of the depth below half the breadth would, clearly, cause a rapid loss of economy in a roofed tank, for the sheeting proper becomes less efficient while the roof area increases as the square of the breadth—*i. e.* inversely as the depth. Increased depth, over a considerable range, is found to cause a relatively slight loss of economy in the whole area, but it should be noted that the loss in the sheeting proper is more than appears from the direct results, the roof area becoming less in relation to the whole. The most important thing to notice, however, is that appreciable departures may be made from the proportions which give the least shell area for a specified volume, without much increase in the area. Labour may often be saved by such means, and the cost of a tank actually reduced in consequence. As an instance, suppose the most economical depth for a certain proposed

tank appeared, from application of the appropriate equation, to be about 6 ft. Now, plates of such width are difficult to obtain, and adherence to the stated depth would usually entail either some less advantageous arrangement of available plates, or the use of narrower plates in two strakes—both involving additional seams for riveting. If the depth were reduced to 5 ft., stock plates might be used, and the additional cost due to the slight increase in area

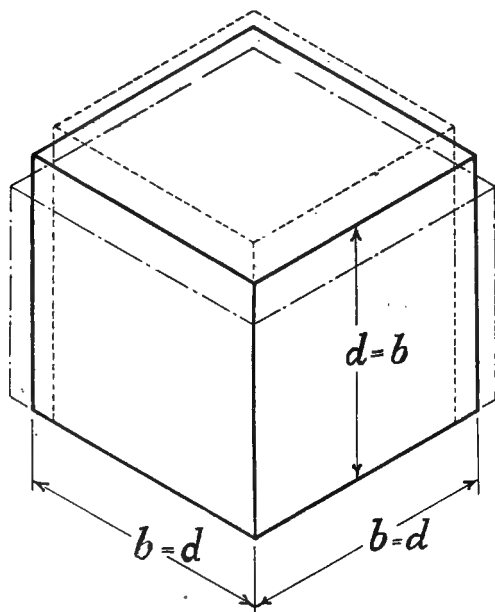


FIG. 8.

of the sheeting would be more than nullified by the reduction in riveting.

Where the roof for a tank is to be of such form and construction that it will cost more (or less) per square foot than the sheeting proper, equation (3) cannot give dependable results. It is, however, a simple matter to obtain a suitable relation for such cases, by introducing the ratio borne by the cost of the roof per unit area to that of the sheeting proper.

Knowing the specification or conditions in any particular case,

it is always possible to form some estimate as to the probable cost of the roof per square foot (including all necessary bearers, purlins, or other supporting work, holding-down bolts, gutters, down-pipes, etc.) as compared with the probable average cost per square foot of the sheeting in the walls and floor. Such particulars can only be estimated, of course, but with intelligent application of the results of experience, the estimates may be made reasonably correct forecasts of the subsequent facts. In cases where some degree of refinement is desired, the estimates may be used to obtain a first set of proportions, and a sketch design prepared, to include all essentials, on that basis. A reliable estimate may then be made as to the relative costs of the roof and sheeting proper (the average for the latter) per square foot, and a more accurate relation obtained for the most economical proportions of the tank.

In stating the conditions symbolically, the alteration in cost per unit area may be taken account of by assuming that no such alteration in cost occurs, but the area to be covered by the roof varied in the same ratio as the cost is actually varied. For instance, if a roof were likely to cost twice as much per square foot as the shell proper, we might imagine the roof made up of a double layer of ordinary sheeting, the total cost being the same, of course.

Suppose a tank is to have a cheap form of roof, the total cost of which per square foot is estimated at half that of the walls and floor; the cost of the tank and roof complete would be the same as though the cost of the roof per unit area were exactly equal to that of the shell proper, but the roof covered only one-half of the tank on plan. The equivalent area of sheeting would then be made up of the four walls, the floor, and half the area to be covered by the roof—

$$A = 4 b d + 1.5 b^2 \quad . \quad . \quad . \quad . \quad . \quad (6)$$

Inserting the value of d from equation (1), which still holds—

$$A = \frac{4 V}{b} + 1.5 b^2,$$

and differentiating with respect to b , regarding V as constant—

$$\frac{d A}{d b} = - \frac{4 V}{b^2} + 3 b,$$

whence, for minimum area of sheeting—

$$4 \text{ V} = 3 b^8 = 4 b^2 d,$$

and—

$$b = \frac{4}{3} (d) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Stating the matter in a general form, suitable for application to any ratio of costs for roofing and shell proper, let r be the ratio borne by the cost of the roof per unit area to that of the shell proper, *i. e.*—

$$r = \frac{\text{cost of roof per unit area}}{\text{cost of walls and floor per unit area}} \quad \cdot \quad (8)$$

Then—

$$A = 4 b d + b^2 (1 + r) \quad . \quad . \quad . \quad . \quad (9)$$

$$= \frac{4 V}{b} + b^2 (1 + r).$$

Keeping V constant, and differentiating with respect to b —

$$\frac{dA}{db} = -\frac{4V}{b^2} + 2b(1+r),$$

whence, for minimum area of sheeting—

$$4 V = 2 b^3 (1 + r) = 4 b^2 d,$$

and—

$$b = d \left(\frac{2}{1 + r} \right) \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Clearly, if $r = 1$ (*i. e.* the cost per unit area is constant for the roof and shell proper), $b = d$, as in equation (5), but whatever value may be assigned to r , equation (10) will give the most economical proportions for a tank square on plan under those circumstances, as regards area of sheeting.

If it be desired to take into account a difference between the costs per unit area of the bottom (as well as the roof) and side sheeting, the ratio of these two costs might be represented by q , just as r is used in equation (8) for the roof.

Equation (9) would then become—

$$\begin{aligned} A &= 4 b d + b^2(q + r) \quad . \quad . \quad . \quad . \quad (9a) \\ &= \frac{4 V}{h} + b^2(q + r) \end{aligned}$$

Keeping V constant, and differentiating with respect to b —

$$\frac{dA}{db} = -\frac{4V}{b^2} + 2b(q+r),$$

whence, for minimum area of sheeting—

$$4V = 2b^3(q+r) = 4b^2d,$$

and—

$$b = d\left(\frac{2}{q+r}\right) \quad \dots \quad (10a)$$

22.—*Tanks Rectangular on Plan.*—Tanks of the form under discussion are often rectangular on plan, instead of square, and it will therefore be well, before passing on to the consideration of other forms, to investigate the relation between changes in the proportions and the consequent alterations in the shell surface for such cases.

In order to avoid the trouble and complication of the work involved in dealing with three variables, let us suppose that the breadth is equal to the depth. The shape will then be as indicated in Fig. 9, the dotted lines showing the variations in the proportions.

For a tank having no roof—

$$V = ld^2, \quad \dots \quad (11)$$

and the area of shell surface—

$$A = 3ld + 2d^2, \quad \dots \quad (12)$$

where l = length of tank, and d = depth = breadth.

From (11), $l = \frac{V}{d^2}$, and, inserting this value for l in (12)—

$$A = \frac{3V}{d} + 2d^2.$$

Differentiating with respect to d , and regarding V as constant—

$$\frac{dA}{dd} = -\frac{3V}{d^2} + 4d,$$

whence, for minimum area of sheeting—

$$4d^3 = 3V = 3ld^2,$$

and—

$$l = \frac{4}{3}d \quad \dots \quad (13)$$

Comparing this result with the 300 sq. ft. obtained for the unroofed tank square on plan, it will be seen that the most economically proportioned tank rectangular on plan requires about 3·7 per cent. more area of sheeting than does the most economically proportioned tank of the same capacity square on plan, both tanks having no roof.

$$\text{If } d = 6 \text{ ft.}, l = \frac{500}{36} = 13\cdot9 \text{ ft.}, \text{ and } A = 250\cdot2 + 72 = 322\cdot2 \text{ sq. ft.}$$

$$\text{If } d = 5 \text{ ft.}, l = \frac{500}{25} = 20 \text{ ft.}, \text{ and } A = 300 + 50 = 350 \text{ sq. ft.}$$

$$\text{If } d = 8 \text{ ft.}, l = \frac{500}{64} = 7\cdot8 \text{ ft.}, \text{ and } A = 187\cdot2 + 128 = 315\cdot2 \text{ sq. ft.}$$

$$\text{If } d = 9 \text{ ft.}, l = \frac{500}{81} = 6\cdot2 \text{ ft.}, \text{ and } A = 167\cdot4 + 162 = 329\cdot4 \text{ sq. ft.}$$

Thus, by comparison of these results with those of the example in Article 21, it will be seen that the increases in area vary from about 4 to 10 per cent., except the case where the length is four times the depth, in which the increase is nearly 17 per cent. It is quite possible, however, that this latter case might, in spite of the relatively large increase in the shell surface, give a cheaper tank than the others, for the walls and bottom could be made, in it, of plates 5 ft. in width, requiring no seams other than those where the walls meet the floor and each other, whereas most of the other cases would involve additional seams and riveting.

For a tank of this shape having a roof, and using r to denote the ratio of costs per unit area for roof and shell proper, as in equation (8)—

$$\begin{aligned} A &= l d (3 + r) + 2 d^2 \quad . \quad . \quad . \quad . \quad (16) \\ &= \frac{V}{d} (3 + r) + 2 d^2 \end{aligned}$$

Differentiating with respect to d , as before—

$$\frac{dA}{dd} = -\frac{V}{d^2} (3 + r) + 4d,$$

whence, for minimum area of sheeting—

$$d^3 \left(\frac{4}{3 + r} \right) = V = l d^2,$$

and—
$$l = d \left(\frac{4}{3} + r \right) \quad . \quad . \quad . \quad . \quad . \quad (17)$$

If $r = 1$ —i. e. the cost per unit area constant throughout,— $l = d$, and the form giving maximum economy of sheeting is (as might be expected from the foregoing considerations) a cube.

There is no need to work further examples regarding the effect of variations from the cubical proportions, for those relating to the tank square on plan will apply.

With any particular value of r appropriate to an individual case, the ideal proportions (as regards economy of shell area) may be calculated from equation (17), and it will be found that, as in the other cases investigated, considerable departure from those proportions may be made when necessary or desirable, without much increasing the area of the sheeting.

If it be desired to take into account a difference between the costs per unit area of the bottom and side sheeting also, let the ratio of these two costs be represented by q , just as r is used for the roof.

Then, equation (16) would become—

$$\begin{aligned} A &= l d (2 + q + r) + 2 d^2 \quad . \quad . \quad . \quad . \quad (16a) \\ &= \frac{V}{d} (2 + q + r) + 2 d^2 \end{aligned}$$

Differentiating with respect to d , as before—

$$\frac{dA}{dd} = -\frac{V}{d^2} (2 + q + r) + 4d,$$

whence, for minimum area of sheeting—

$$d^3 \left(\frac{4}{2 + q + r} \right) = V = l d^2,$$

and—

$$l = d \left(\frac{4}{2 + q + r} \right) \quad . \quad . \quad . \quad . \quad (17a)$$

A long, narrow, and shallow tank covers more ground than one of the same capacity more nearly approaching the cubical form, and this is sometimes an important matter; sometimes, again, a long, narrow tank can be conveniently placed over an existing building, whereas a square plan would encroach upon valuable yard space. All these matters, as well as many others of a similar

nature, must be considered in each case before any particular shape or set of proportions can be laid down as the most economical.

23.—*Cylindrical Tanks*.—To examine the cylindrical form as regards area of sheeting for cubic capacity, suppose the diameter is d , and the height h , the bottom and roof (if any) being considered first as flat and costing the same per unit area as the side sheeting. Then the volume, or capacity, will be—

$$V = 0.7854 d^2 h, \quad . \quad . \quad . \quad . \quad . \quad (18)$$

and the area of shell surface, with no roof—

$$A = 3.1416 d h + 0.7854 d^2 \quad . \quad . \quad . \quad . \quad (19)$$

From (18), $h = \frac{V}{0.7854 d^2}$, and inserting this value for h in (19)—

$$A = \frac{3.1416 d V}{0.7854 d^2} + 0.7854 d^2 = \frac{4V}{d} + 0.7854 d^2$$

Differentiating with respect to d , and regarding V as constant—

$$\frac{dA}{dd} = -\frac{4V}{d^2} + 1.5708 d,$$

whence, for minimum area of sheeting—

$$1.5708 d^3 = 4V = 3.1416 d^2 h,$$

and

$$h = \frac{d}{2} \quad . \quad . \quad . \quad . \quad . \quad (20)$$

Inserting this value for h in equations (18 and (19)—

$$V = 0.3927 d^3 \quad . \quad . \quad . \quad . \quad . \quad (21)$$

and

$$A = 2.3562 d^2 \quad . \quad . \quad . \quad . \quad . \quad (22)$$

The proportions just deduced are indicated, for convenient reference, in Fig. 10.

For a volume of 500 cub. ft. (chosen so that the results obtained may be comparable with those of the preceding examples for rectangular tanks)—

$$d^3 = \frac{500}{0.3927} = 1273.2 \text{ cub. ft.},$$

whence—

$$d = \sqrt[3]{1273.2} = 10.83 \text{ ft.}, \text{ and } h = \frac{d}{2} = 5.42 \text{ ft.}$$

Then—

$$A = 2.3562 d^2 = 2.3562 \times 117.3 = 276.4 \text{ sq. ft.}$$

Comparing this result with the 300 sq. ft. required for the unroofed tank square on plan, there is a saving of 23.6 sq. ft. in 300, or about 7.87 per cent. in the area of sheeting, but to set against this there is the additional cost of cutting the bottom plates to the circular curve, setting out the riveting for the circumference of these plates (and for the bottom curb angle) on curved pitch lines, and bending the side plates to give the cylindrical form. Also, both bottom and side plates for cylindrical tanks are more awkward and troublesome to handle, and require much more space in vehicles for haulage and transport, than do flat rectangular plates.

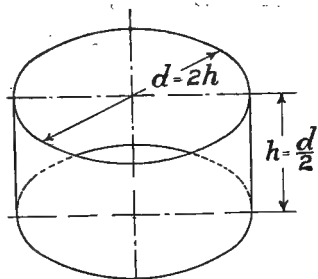


FIG. 10.

To investigate the effects of departure from the proportions of equation (20), giving the least area of sheeting for this case, let us consider four examples, two having the diameter more, and two less, than 10.83 ft., the volume being kept constant at 500 cub. ft.

$$\begin{aligned} \text{If } d = 12 \text{ ft., } h &= \frac{500}{0.7854 \times 144} = 4.42 \text{ ft., and } A \\ &= 166.7 \times 113.1 = 279.8 \text{ sq. ft.} \end{aligned}$$

$$\begin{aligned} \text{If } d = 13 \text{ ft., } h &= \frac{500}{0.7854 \times 169} = 3.77 \text{ ft., and } A \\ &= 153.9 \times 132.7 = 286.6 \text{ sq. ft.} \end{aligned}$$

$$\begin{aligned} \text{If } d = 10 \text{ ft., } h &= \frac{500}{0.7854 \times 100} = 6.37 \text{ ft., and } A \\ &= 200.1 + 78.5 = 278.6 \text{ sq. ft.} \end{aligned}$$

$$\begin{aligned} \text{If } d = 9 \text{ ft., } h &= \frac{500}{0.7854 \times 81} = 7.87 \text{ ft., and } A \\ &= 222.7 + 63.5 = 286.2 \text{ sq. ft.} \end{aligned}$$

Hence, a variation (either increase or decrease) of nearly 20 per cent. from the diameter giving the minimum area of sheeting, causes

an increase of only 3·69 per cent. in the area of the shell. Full advantage should be taken of this fact to secure the most economical proportions for each particular tank, for by regulating the height (within reasonable limits, of course) to suit convenient widths of plates, it is often possible to save one complete circumferential seam, and several vertical ones, by making the number of strakes one less than would otherwise be necessary.

With a flat roof costing the same per unit area as the side sheeting, V will be as before, but the area of shell surface will be—

$$A = 3\cdot1416 \, d \, h + 1\cdot5708 \, d^2 \quad . \quad . \quad . \quad (23)$$

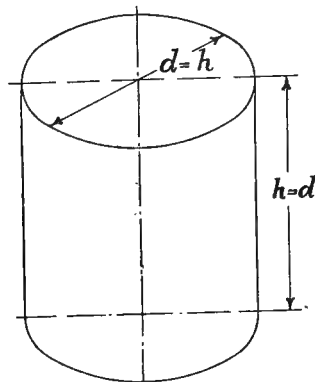


FIG. 11.

Inserting in (23) the value of h from (18)—

$$A = 4 \frac{V}{d} + 1\cdot5708 \, d^2.$$

Differentiating with respect to d , and regarding V as constant—

$$\frac{dA}{dd} = -\frac{4V}{d^2} + 3\cdot1416 \, d,$$

whence, for minimum area of sheeting—

$$3\cdot1416 \, d^3 = 4V = 3\cdot1416 \, d^2 \, h,$$

and, obviously—

$$h = d \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

These proportions are, like those for other cases, indicated in a diagram for easy reference, the illustration here being Fig. 11.

Inserting the value for h from (24) in equations (18) and (23)—

$$V = 0.7854 d^3 \quad . \quad . \quad . \quad . \quad . \quad (25)$$

and

$$A = 4.7124 d^2 \quad . \quad . \quad . \quad . \quad . \quad (26)$$

For a volume of 500 cub. ft. (chosen again for the purpose of comparison)—

$$d^3 = \frac{500}{0.7854} = 636.6 \text{ cub. ft.}, \text{ whence } d = \sqrt[3]{636.6} = 8.6 \text{ ft.}$$

Then, $A = 4.7124 d^2 = 4.7124 \times 73.96 = 348.7$ sq. ft., of which 58.1 sq. ft. will be in the roof, and 290.6 sq. ft. in the shell proper.

Comparing this with the roofed tank square on plan, which required 63 sq. ft. for the roof and 315 sq. ft. for the shell proper, giving a total area of 378 sq. ft., the roofed cylindrical form shows a saving of 29.3 sq. ft. in 378, or about 7.7 per cent. of sheeting surface, to set against the extra cost of labour and transport for the material of the cylindrical tank.

It is well to investigate the effects of departure from the proportions $h = d$, of equation (24), keeping the volume constant at 500 cub. ft.

$$\begin{aligned} \text{If } d = 10 \text{ ft.}, h &= \frac{500}{0.7854 \times 100} = 6.37 \text{ ft.}, \text{ and } A \\ &= 200.1 + 157.0 = 357.1 \text{ sq. ft.}, \end{aligned}$$

of which 78.5 sq. ft. will be in the roof, and 278.6 sq. ft. in the shell proper.

$$\begin{aligned} \text{If } d = 11 \text{ ft.}, h &= \frac{500}{0.7854 \times 121} = 5.26 \text{ ft.}, \text{ and } A \\ &= 181.9 + 190.1 = 372.0 \text{ sq. ft.}, \end{aligned}$$

of which 95.0 sq. ft. will be in the roof, and 277.0 sq. ft. in the shell proper.

$$\begin{aligned} \text{If } d = 7 \text{ ft.}, h &= \frac{500}{0.7854 \times 49} = 12.99 \text{ ft.}, \text{ and } A \\ &= 285.9 + 77.0 = 362.9 \text{ sq. ft.}, \end{aligned}$$

of which 38.5 sq. ft. will be in the roof, and 324.4 sq. ft. in the shell proper.

$$\begin{aligned} \text{If } d = 6 \text{ ft.}, h &= \frac{500}{0.7854 \times 36} = 17.68 \text{ ft.}, \text{ and } A \\ &= 333.4 + 56.5 = 389.9 \text{ sq. ft.}, \end{aligned}$$

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of which 28.3 sq. ft. will be in the roof, and 361.6 sq. ft. in the shell proper.

Hence, increasing the diameter by 27.9 per cent. increases the total shell surface by only 6.7 per cent., and reducing the diameter by 30.2 per cent. increases the area of the sheeting by 11.8 per cent.

It will be seen from the calculations that as the tank is made shallower, the sheeting proper (in the cylindrical wall and floor) becomes more efficient until the proportion $d = 2h$ of equation (20) is reached, after which, further reduction in the depth would cause a rapid falling off in the economy of sheeting area. Increasing the depth causes a relatively greater loss of economy, and this loss is, to a large extent, a loss of efficiency in the sheeting proper.

A point worthy of notice in passing is the fact that, so far as regards economy of sheeting *area* only, the loss in an unroofed tank (rectangular or cylindrical) is most marked as the tank becomes shallower—*i. e.* more slab-like; and in a roofed tank as the tank becomes deeper—*i. e.* more rod-like. As has been shown, there are other and important considerations to be taken into account, but this point is of real interest and importance, and we shall have more to say concerning it presently.

The roofs of cylindrical tanks are nearly always made convex (the surface being usually spherical), with the object of throwing off rain, condensation water, and other corrosive solutions. The roof sheeting is usually quite thin—14 B.W.G. being a commonly used thickness—and is carried upon light-framed trusses spanning from side to side, or radiating from a central post to the circumference. It is often advisable to countersink all rivets in the roof sheeting at the outer surface, thus reducing the liability to corrosive actions.

The actual area of the roof sheeting may, then, be appreciably more than that of a horizontal section through the tank, and the extra work in dishing the plates, combined with the cost of the supporting trusses, and operating against the effects of using lighter sheeting, may make the cost of the complete roof per square foot either more or less than the average cost per square foot of the shell proper. To allow for this in determining the proportions which will give maximum economy as regards the area of sheeting, let r denote the ratio of the costs per unit area for the roof and shell

proper, as in equation (8), and let the altered cost be allowed for as though it were an alteration in the effective area at uniform cost.

Then, equation (23) becomes—

$$\begin{aligned} A &= 3.1416 \, d \, h + 0.7854 \, d^2 (1 + r) \quad . \quad . \quad (27) \\ &= \frac{4V}{d} + 0.7854 \, d^2 (1 + r). \end{aligned}$$

Differentiating with respect to d , and regarding V as constant—

$$\frac{dA}{dd} = -\frac{4V}{d^2} + 1.5708 \, d (1 + r),$$

whence, for minimum effective area of sheeting—

$$4V = 1.5708 \, d^3 (1 + r) = 3.1416 \, d^2 \, h,$$

and

$$h = d \left\{ \frac{1+r}{2} \right\} \quad . \quad . \quad . \quad . \quad . \quad (28)$$

Clearly, if $r = 1$ (*i. e.* the cost of the complete roof per unit area of projected surface, or of ground covered, equal to that of the actual containing surfaces on the average), $h = d$, which agrees with equation (24).

Equation (28) should be compared with equation (10) for a tank square on plan with a roof, and their similarity noted.

If it be desired to take into account a difference between the costs per unit area of the bottom and side sheeting, the ratio of these two costs might be represented by q , just as r is used for the roof in preceding cases. Equation (28) would then become—

$$h = d \left\{ \frac{q+r}{2} \right\}, \quad . \quad . \quad . \quad . \quad . \quad (29)$$

from which it is clear that if the bottom and roof both cost less per unit area than does the side sheeting, the proportions for least area of sheeting will give h less than d , and vice versa.

Horizontal cylindrical tanks, like boiler shells in appearance, are sometimes used where a comparatively small capacity for storage is required, and for the carriage of liquids on railway trucks. These would provide a typical instance for the application of equations (23) to (26) inclusive, or, if the ends be dished, equations (28) and (29). For railway purposes, however, considerations of limiting

over-all dimensions are of more importance than that of economy in shell surface, and the latter is therefore sacrificed. In other circumstances, however, the facts deduced above are at least worth considering.

Cylindrical tanks, though economical in many ways, sometimes have the disadvantage of occupying more ground surface than would be required for a square or rectangular tank, because of the awkwardly shaped corner pieces left when a cylindrical tank is fitted into a rectangular space not much more in diameter than the tank. Where there is plenty of space, however—as there usually is on the roof of a building, for instance,—they may be made a very efficient form of construction, and should be used more freely than they are.

One important advantage is possessed by cylindrical tanks as compared with the rectangular form, in that the cylindrical side plates need no internal staying. The ordinary theory for “thin cylinders” is used, as will be explained later, for their design, and although it might appear that the side sheeting of a tank (say) 100 ft. in diameter is practically flat, the results of experience seem to indicate that the theory is justified, even for diameters much larger than that instanced, the liquid pressure internally causing the plates of each strake to be stressed in tension, with no appreciable bending action except such as is due to secondary stresses. Here and there one comes across a tank in which the designer, evidently feeling doubtful, has provided some sort of staying, but it is frequently found that the stays are so arranged and proportioned that they could not be of any real assistance in stiffening the side plates against bending.

24.—*Cylindrical Tanks with Dished Bottoms.*—We have not yet considered the question as to what is most likely to be the most suitable shape for the dished bottom to a cylindrical tank from the point of view of stress limitation; but some idea as to the economy of such bottoms, as regards area of sheeting, may be gathered from a consideration of a hemispherical and a conical bottom, observing the influence of variations in the apex angle in the latter case, and ignoring the effects of bending stresses in the plates.

First, in the case of a hemispherical bottom, as indicated in Fig. 12, if d be the diameter of the cylindrical tank, the capacity

of the bottom will be $V_b = d^3 \left(\frac{\pi}{12} \right) = 0.2618 d^3$, and the area of the sheeting $A_b = d^2 \left(\frac{\pi}{2} \right) = 1.5708 d^2$. So, by doubling the area of the bottom sheeting, as compared with the flat-bottomed tank, additional storage capacity of $0.2618 d^3$ is obtained with the hemispherical bottom, and the joist system for supporting the flat bottom is avoided. To set against this saving, however, there is the extra cost of dishing the plates, increased and more difficult riveting and caulking, and curved girders to carry the tank.

For obvious reasons, dished bottoms are used on elevated tanks only.

A convenient method for comparing the storage efficiency of these dished bottoms is to determine the depth of the liquid they contain, assuming that liquid uniformly distributed over the sheeting surface. Thus, with the hemispherical bottom, the capacity is $0.2618 d^3$, and the area of sheeting $1.5708 d^2$. If

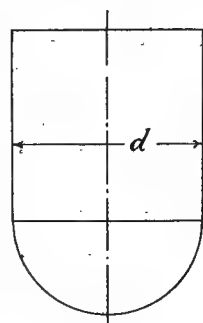


FIG. 12.

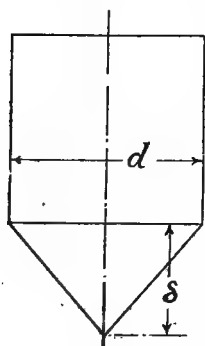


FIG. 13.

the sheeting could be laid out to form a flat surface, and liquid having a volume equal to the capacity of the bottom were spread over the flat surface, being supported at the sides by auxiliary walls (or frozen to a solid), the liquid would form a slab, of depth $D_s = \frac{0.2618 d^3}{1.5708 d^2} = \frac{d}{6}$.

This depth D_s (the suffix *s* indicating that it relates to the spherical bottom) might well be termed the "effective storage depth of the sheeting," and it may be employed with advantage in comparing not only dished bottoms, but also tanks themselves, of different forms and proportions.

With a conical bottom, as in Fig. 13, the capacity of the bottom will be $V_b = \frac{\pi d^2 \delta}{12} = 0.2618 d^2 \delta$, where δ is the "drop" of the cone, as indicated in the sketch. The area of the sheeting will be

$A_b = \frac{\pi d}{2} \sqrt{\delta^2 + \frac{d^2}{4}} = 0.7854 d \sqrt{4\delta^2 + d^2}$. If $\delta = \frac{d}{2}$ (as for the hemispherical bottom), $V_b = 0.1309 d^3$ —i. e. just half that for the hemisphere,—and $A_b = 0.7854 d^2 \sqrt{2}$, whence the “effective storage depth” $D_c = \frac{0.1309 d^3}{0.7854 d^2 \sqrt{2}} = \frac{d}{6 \sqrt{2}} = \frac{d}{8.48}$.

If $\delta = d$, then $V_b = 0.2618 d^3$, and $A_b = 0.7854 d^2 \sqrt{5}$, whence $D_c = \frac{d}{6.71}$.

If $\delta = 1.5 d$, then $V_b = 0.3927 d^3$, and $A_b = 0.7854 d^2 \sqrt{10}$, whence $D_c = \frac{d}{6.32}$.

If $\delta = 2 d$, then $V_b = 0.5236 d^3$, and $A_b = 0.7854 d^2 \sqrt{17}$, whence $D_c = \frac{d}{6.18}$.

If $\delta = 3 d$, then $V_b = 0.7854 d^3$, and $A_b = 0.7854 d^2 \sqrt{37}$, whence $D_c = \frac{d}{6.08}$.

The last three (certainly the last two) instances are inadmissible for practical purposes, but they serve to illustrate the comparison of “sheeting efficiencies” for the two forms. The proportion $\delta = d$ would be fairly suitable for practice, and then, it should be noticed, the capacity would be equal to that for the hemispherical bottom, while the area of the sheeting would be $1.7562 d^2$, as against $1.5708 d^2$ for the hemisphere—not a very large difference in actual area with tanks of such diameters as are likely to be suitable for dished bottoms, while the labour in bending the plates, and also in riveting and caulking, would be less with the cone than with the hemisphere. Moreover, the difference in area for an actual case would probably be less than that shown by calculations on the above basis, for the bottom (whether hemispherical or conical), with the methods of construction usually employed, could not well spring from the cylindrical wall of the tank. If it did, some form of external bracketed ring construction would be necessary, to provide a seating for the transmission of the entire weight to the curved supporting girders, and this would be both costly and unsightly. It is usual to place the

supporting girders beneath the cylindrical wall, or sometimes, even a few inches nearer to the centre of the tank, and the dished bottom then springs from the inner edge of the girder ring; hence, the d for the bottom will be less than the diameter of the cylinder.

For a cylindrical tank having some form of roof and a hemispherical bottom, as indicated in Fig 12, the general investigation for economy of form might be as follows. Adopting the symbols used in the preceding cases, and denoting the height of the cylindrical portion by h —

$$V = \frac{\pi}{4} d^2 h + \frac{1}{2} \left(\frac{4}{3} \pi \frac{d^3}{8} \right) = \frac{\pi}{4} d^2 h + \frac{\pi}{12} d^3;$$

whence—
$$\frac{\pi}{4} d^2 h = V - \frac{\pi}{12} d^3,$$

and—
$$h = \frac{4}{\pi} \frac{V}{d^2} - \frac{d}{3}.$$

$$A = \pi d h + \frac{\pi}{4} d^2 r + \frac{1}{2} (\pi d^2 q)$$

Substituting the value of h from above—

$$A = \frac{4V}{d} - \frac{\pi d^2}{3} + \frac{\pi}{4} d^2 r + \frac{\pi}{2} d^2 q.$$

Differentiating with respect to d , and regarding V as constant—

$$\frac{dA}{dd} = -\frac{4V}{d^2} - \frac{2\pi d}{3} + \frac{\pi d r}{2} + \pi d q.$$

For minimum area of sheeting, $\frac{dA}{dd} = 0$; whence—

$$\begin{aligned} \frac{4V}{d^2} &= \frac{\pi d r}{2} + \pi d q - \frac{2\pi d}{3} \\ \therefore \pi h + \frac{\pi d}{3} &= \frac{\pi d r}{2} + \pi d q - \frac{2\pi d}{3} \end{aligned}$$

So that—
$$h = d \left(\frac{r}{2} + q - 1 \right). \quad (30)$$

It should be noted that, in the foregoing investigation, the area of the roof has been taken as that of a horizontal section of the tank proper. The object of this is to leave the result applicable to any

form of roof, and it is only necessary to observe that the value employed for r must take into account any increase in area for the roof beyond that of the tank section—in other words, r must be evaluated on the basis of roof-cost per square foot of actual tank (horizontal) section.

For a cylindrical tank having some form of roof and a conical bottom, as indicated in Fig. 13, the investigation for economy of form might be as follows—

$$V = \frac{\pi}{4} d^2 h + \frac{1}{3} \left(\frac{\pi}{4} d^2 \delta \right);$$

whence—
$$h = \frac{4V}{\pi d^2} - \frac{\delta}{3}$$

$$A = \pi d h + \frac{\pi}{4} d^2 r + \frac{\pi d q}{4} \sqrt{d^2 + 4 \delta^2}$$

Substituting the value of h from above, and writing $k d$ instead of δ —

$$A = \frac{4V}{d} - \frac{\pi k d^2}{3} + \frac{\pi}{4} d^2 r + \frac{\pi}{4} d^2 q \sqrt{1 + 4 k^2}$$

Differentiating with respect to d , and regarding V as constant—

$$\frac{dA}{dd} = -\frac{4V}{d^2} - \frac{2\pi k d}{3} + \frac{\pi d r}{2} + \frac{\pi d q \sqrt{1 + 4 k^2}}{2}$$

For minimum area of sheeting, $\frac{dA}{dd} = 0$; whence—

$$\frac{4V}{d^2} = \pi \left(\frac{dr}{2} + \frac{d q \sqrt{1 + 4 k^2}}{2} - \frac{2 k d}{3} \right);$$

$$\therefore \pi \left(h + \frac{k d}{3} \right) = \pi \left(\frac{dr}{2} + \frac{d q \sqrt{1 + 4 k^2}}{2} - \frac{2 k d}{3} \right);$$

whence—
$$h = d \left(\frac{r}{2} + \frac{q \sqrt{1 + 4 k^2}}{2} - k \right);$$
$$= d \left(\frac{r + q \sqrt{1 + 4 k^2} - 2 k}{2} \right). \quad \dots \quad (31)$$

Here, again, the value employed for r must be that obtained on the basis of roof-cost per square foot of actual tank (horizontal) section.

To judge from the remarks of the few writers who have dealt with dished bottoms for tanks, appearance would seem to be the most important (if not the only) factor in their design. This, of course, is by no means the case. In some circumstances the stand-pipe or water tower is an important public structure, and is so placed as to form a very prominent and noticeable object; and appearance is of considerable importance in such cases, though the efforts of some designers to make water towers "pleasing" are rather amusing—to those who are not called upon to bear the cost.

The appearance of such a structure depends largely upon its altitude, position, dimensions, and proportions, and it will be shown later that ugliness may be avoided by considering these matters, rather than by adopting one particular form for all conditions and circumstances. The ordinary and obvious principles of natural growth and formation are the best guide to follow in these and similar points, and it cannot be too strongly urged that the first principle of beauty in architecture is true suitability to purpose. It is a pity that engineers do not pay more attention to such simple and interesting matters, for there is really no reason why a tank, or any other strictly utilitarian structure, should be offensive to the eye and a reproach to the profession. Nothing need be sacrificed in design which could make for efficiency in working, or the minimising of the costs of manufacture and erection. Indeed, there is good reason to believe that a structure proportioned on the lines of natural growth will of necessity be economical and efficient, provided the application be made intelligently.

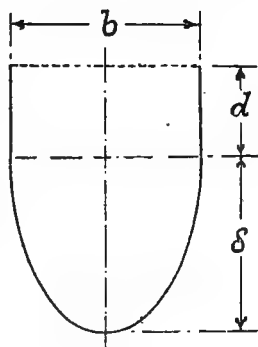


FIG. 14.

25. *Rectangular Tanks with Trough Bottoms.*—A form which, though seldom used, might well be considered where the conditions are favourable, is a rectangular tank with a trough bottom, as indicated in Fig. 14, which shows a cross-section of the tank and trough. If carried on stanchions or piers, the side walls of the tank may be arranged to act as the girders for transmitting the loads between the supports. Such a form would evidently be suitable for elevated

tanks in which, owing to circumstances of available space, the length must be great as compared with the width. The plates forming the trough could be easily bent to the required shape, and the erection and riveting would be simple. With vertical flat ends to the trough, it would probably be necessary to provide stiffening for the plates, against the pressure of the contained liquid, but this is a simple matter.

The trough might be semi-elliptical in cross-section, or some convenient modification of that curve might be adopted. A semi-cylindrical trough would give the greatest capacity as compared with the area of sheeting, but, as will be shown presently, might cause more bending actions in the plates than would the semi-elliptical shape, owing to the greater depth of the liquid at the middle of the trough.

It should be noticed that such a trough cannot be properly regarded as "flexible." To consider a strip of the trough-sheeting between two sections at right angles to the length of the tank, and treat it as though it were a chain, with loads applied at the points of contact between the links, is to make an assumption which cannot be realised. A chain would be free to take up that shape imposed upon it by the loading, with no resistance other than the slight friction between the links, whereas a solid steel plate, of such thickness as is practicable for tank work, is not free to change its shape, and any tendency to cause such change of shape would induce more or less severe stresses in the material by reason of the bending action set up. Moreover, with a chain, variations in the loading would cause changes in the shape of the chain, whereas, in a trough-bottomed tank, although the loading may vary both widely and frequently through alterations in the head of the contained liquid, if appreciable changes of shape occurred in the riveted plates forming the trough, the seams would not remain tight against leakage. This, of course, applies equally to all curved or dished bottoms. Probably the best method for designing in such cases is to determine the shape which will give the least bending actions in the curved plates under the greatest pressures, and then to provide such stays and framing as will prevent undue stresses and excessive changes of shape with the different conditions of loading possible or likely to occur.

The significance of this will be clear on reference to Fig. 15, which

represents the cross-section of a trough-bottomed rectangular tank. Suppose the plates forming the trough are so thin as to be perfectly flexible, and that, with the tank full, they take the shape indicated in the sketch (a). Now, if the liquid fell to the level A A, as at (b), the portions of the sheeting A B would become straight, and it is clear that riveted seams—particularly circumferential seams—subjected to such changes of shape could not be relied upon to remain tight. Plates of ordinarily practicable thickness would, to some extent, resist this tendency to change of shape, and thus stresses due to the bending action would be added to the direct tensile stress. As the loading decreases in magnitude, the direct tensile stress in the plates is, of course, reduced, and the object should be to so

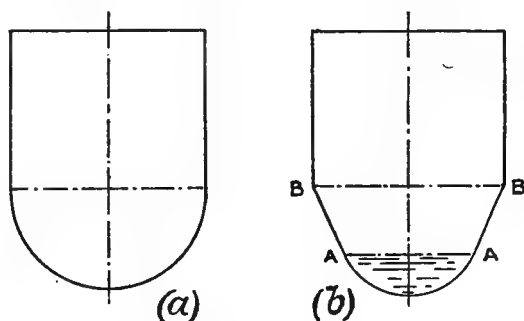


FIG. 15.

arrange that the additional stresses due to bending, under a reduced loading, do not more than counterbalance the corresponding reductions in the direct tensile stresses.

In Fig. 15 an exceptional case has been chosen, solely for the purpose of illustrating clearly the effects of such changes in shape, and the change is exaggerated. This affects the point in degree only, however, and not in principle, for although the effects may be reduced in magnitude by a judicious choice of the most suitable shape for the trough (or dished) bottom, they cannot be eliminated.

Here an important point of difference between rectangular and cylindrical tanks having trough or dished bottoms should be noticed. In the rectangular tank of Fig. 15 (a), the contained liquid in the tank proper will cause an outward horizontal thrust to act

upon both the side walls, and some provision for resisting these thrusts is necessary to ensure stability. With a cylindrical tank no need for such additional provision arises, because the outward pressures at each level act uniformly in directions radiating from the centre of the tank, and the outward thrusts are resisted by simple circumferential tensions in the sheeting. Further, when the liquid fills only a portion of the trough in a rectangular tank, as at (b) in Fig. 15, there will be an inward pull at the bottoms of the side walls, tending to draw the walls together, and although these inward pulls are likely to be of small magnitude in a properly designed tank, they must be provided for.

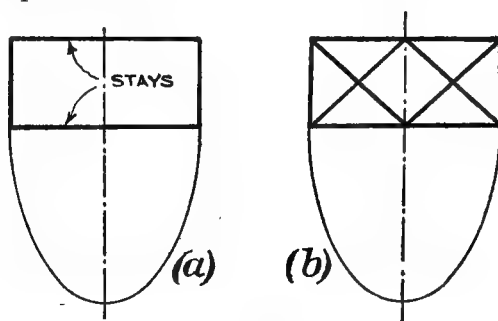


FIG. 16.

The walls of the rectangular tank of Fig. 14 may be supported against horizontal forces by means of transverse stays at the top and bottom, as indicated at (a) in Fig. 16; or, preferably, by framed bracing, as shown at (b) in that sketch. These stays or frames should be spaced at a convenient distance apart, and the walls provided with sufficient strength and stiffness to resist the horizontal forces over this distance. The most suitable distance between the transverse stays or bracings must vary with, and depend upon, such conditions as the height of the side walls; and there will be, for every case, one spacing which is more economical than any other. With transverse stays very close together, the side walls would need little or no additional stiffening, but the cost of the stays might be heavy. On the other hand, with the stays far apart, while the cost of the stays might be trifling, considerable expense would probably be incurred in stiffening the side walls sufficiently to enable them to

properly resist the outward pressures between the stays. To keep down the cost of the stays it is necessary to use as few of them as possible, and to avoid extra expense in stiffening the side walls of the tank it is necessary to use as many lateral stays as possible. These opposing considerations will determine the most economical spacing for the stays, and it should be borne in mind when dealing with such matters that a heavy transverse stay or framed bracing need cost but little more to make and fix than does a light one, the total cost of such stays or bracings in a tank being much more nearly proportional to their number than to their individual or aggregate weights; and also that, by means of the intelligent disposal of material, the side walls of a tank may be so constructed as to possess a reasonable degree of lateral strength and stiffness, instead of practically none, for little or no increase in the cost.

With an ordinary flat-bottomed rectangular tank, the floor acts as a sufficient tie to the side walls at the bottom, stays to the upper parts only of the walls being needed, and whatever increase of cost for lateral staying results from the use of such a form as that indicated in Fig. 14 must be regarded as tending to discount the advantages of a trough instead of a flat bottom, supported on joists—provided, of course, that the staying be designed with proper regard for efficiency and economy.

In tanks, the contained substance being always liquid, the outward thrusts and inward pulls referred to above will always be symmetrical, and hence, there are no unbalanced horizontal forces applied to the structure or its supports from the mere storage of the substance, and other than forces from such external agencies as wind pressure, with an elevated tank of the section shown in Fig. 14. With bunkers and silos, to contain granular material which may stand at comparatively steep angles with the horizontal, unbalanced forces of considerable magnitude may be set up unless means for their prevention—usually by subdividing the storage space into relatively narrow cells—be adopted.

Matters would not be much improved, with the rectangular tank, by the use of a segmental trough, as indicated in Fig. 17, for the curved plates, unless they were of much greater thickness than required for containing the liquid, could not provide adequate support to the side walls against horizontal forces. Moreover,

a segmental trough has two disadvantages as compared with one semi-elliptical in cross-section with the major axis of the ellipse vertical. First, though the cost of the curved sheeting would prob-

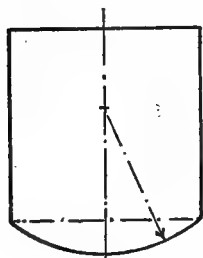


FIG. 17.

ably be only slightly reduced, there would be a very large proportional loss of storage capacity; and, second, the connections of the trough plates to the tank walls would be troublesome and costly to make properly satisfactory and reliable.

For the purpose of investigating the effects of a semi-elliptical trough bottom to a rectangular tank, as regards storage capacity and sheeting surface, it will be sufficient to use the approximate relations for the ellipse—

$$\text{Perimeter} = \frac{\pi}{2} (\text{major axis} + \text{minor axis});$$

$$\text{Area} = \frac{\pi}{4} (\text{major axis} \times \text{minor axis}).$$

If b represents the breadth of the tank (Fig. 14), the minor axis of the ellipse will be b , and the semi-major axis may be conveniently denoted by δ , the "drop" of the trough. Then, taking b , d , and δ in lineal feet, we may calculate for one foot run of the tank and trough, so that areas will be in square feet, and volumes in cubic feet. It should be noticed that this basis for calculation assumes that the length of the tank is so great as compared with its width that the sheeting in the ends is insignificant in comparison with that in the sides. The case for a shorter tank will be considered presently.

Ignoring the question as to whether the tank is to have a roof or not (since the roof would be the same so long as the breadth is unchanged), the area and volume of the combined tank and trough per foot run will be—

$$A = 2d + \frac{1}{2} \{ 1.5708 (b + 2\delta) \} = 2d + 0.7854 (b + 2\delta);$$

$$\begin{aligned} V &= b d + \frac{1}{2} \{ 0.7854 (b \times 2\delta) \} = b d + 0.7854 (b \delta) \\ &= b (d + 0.7854 \delta). \end{aligned}$$

Comparable with these are: $A = 2d + b$, and $V = b d$, for the simple flat-bottomed rectangular tank.

If the volume of the trough is to be equal to that of the tank proper, we shall have : $0.7854 (b \delta) = b d$, whence—

$$\delta = \frac{d}{0.7854} = 1.27 d.$$

Inserting this value for δ in the expression for the area of the sheeting—

$$A = 2 d + 0.7854 (b + 2.54 d) = 2 d + 0.7854 b + 2 d \\ = 4 d + 0.7854 b.$$

Now, to double the volume of the flat-bottomed tank by doubling the depth would require a sheeting area of $4 d + b$, and hence, the trough would give a saving in the area of sheeting equal to

$$(4 d + b) - (4 d + 0.7854 b) = 0.2146 b.$$

This shows the extent to which, with a fixed breadth, the semi-elliptical form is more economical as regards sheeting area, for a given volume, than the rectangular flat-bottomed tank. The semi-elliptical trough has also the advantage that, when full, the plates may be subjected to simple tension, whereas the flat sides of the rectangular form are called upon to resist bending actions, and must, therefore, be either well stayed or of greater thickness. Moreover, supporting joists are required with the flat bottom.

Hence, it would appear that considerable saving may be effected by the use of a trough bottom, and that the saving would be most if the rectangular portion were rejected altogether, the storage capacity being entirely in the trough. If that were done, however, the tank would be difficult to support, and the cost of the additional girders and work rendered necessary would outweigh much of the saving.

Perhaps the best method, where circumstances permit, would be to make the vertical side walls of sufficient depth to form the supporting girders only, putting all the remaining capacity into the trough.

If the trough is to have n times the capacity of the simple tank, the breadth being, of course, the same for both—

$$0.7854 b \delta = n b d, \text{ whence } \delta = d (1.27 n),$$

giving the area of sheeting for the trough—

$$A_s = 0.7854 (b + 2.54 n d) = 2 n d + 0.7854 b.$$

To give the simple tank n times its original volume by increasing the depth would require a plate area of $2 n d + b$, and the saving in sheeting area is again $0.2146 b$.

Apparently, then, the saving in the area of sheeting is independent of the depth d , and hence it might seem that economy is to be secured by making the tank as broad as possible. This is true up to a certain limit only, however, and it will be seen presently that the best results are to be obtained when the breadth and depth are in a particular relation to each other.

It will be well to examine briefly the effects of variations in the proportions of the cross-section for the rectangular trough-bottomed tank, first taking the case in which the length is great as compared with the breadth.

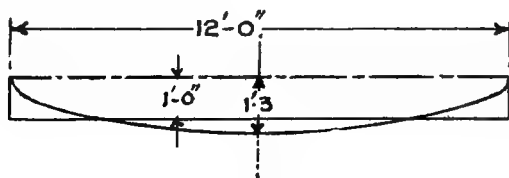


FIG. 18.

From the expressions obtained above, it would appear that economy of sheeting area, with a semi-elliptical trough, is to be obtained by making the breadth as large as possible; but a little consideration will show that this is not true beyond a certain limit. Those expressions were based upon the approximate relation for the perimeter of the ellipse, and this is the reason for the erroneous indication of great breadth as tending to economy of sheeting area. The approximate relation gives reasonably good results where the major and minor axes are not widely different, but cannot be relied upon where the ellipse is long and narrow. A single instance will suffice to show the error of the approximate relation in extreme cases. Consider the cross-section of an unroofed rectangular tank 12 ft. in width and 1 ft. in depth, as indicated in Fig. 18, and compare it with the semi-elliptical trough of equal breadth and capacity also shown. Now, the approximate relation would give the area of sheeting for the trough per foot run as $0.7854 (12 + 2.6) = 11.4$ sq. ft.—i. e. a curved line less in length by one-

twentieth than the straight line between the same terminal points, which is clearly untrue. As the major axis becomes greater and greater as compared with the minor axis, it is obvious that there will, in fact, be less and less difference between the sheeting areas per foot run for the trough and the equivalent rectangular tank of equal width, and the same will be true whether the major axis be horizontal or vertical—the advantage of the trough (as regards sheeting area) being equally lost whether the cross-section be very deep and narrow, or very broad and shallow.

If the question be regarded from the common-sense point of view, it will be clear that the greatest economy of sheeting area, to contain a specified volume per foot run, would be obtained if the tank and trough were combined to form a semi-cylindrical trough. Hence, in selecting the proportions for a particular case, the endeavour should be to obtain a cross-section for the combined tank and trough which differs as little as possible from the equivalent semi-circle.

Since there must be vertical side walls to form the supporting girders, even were the trough bottom semi-cylindrical the combined cross-section would differ from a semi-circle. Moreover, as has been stated already, to minimise bending actions in the curved plates under maximum head, the section of the trough should be semi-elliptical, with the major axis vertical, and therefore, a certain amount of divergence from the semi-circular cross-section is unavoidable for practical reasons. This divergence should, however, be the minimum consistent with practical considerations, and deep, narrow troughs should not be employed unless the circumstances prevent the use of a more economical shape. The broad, shallow trough is, of course, objectionable by reason of its shape being unsuitable for the loading.

A numerical example will provide the best means for obtaining some idea of the extent to which practicable cross-sections involve increased sheeting area as compared with the equivalent semi-circular sections. Take a width constant at 14 ft., and consider three different depths, the capacity of the trough being equal to that of the rectangular portion in each case.

First, let $d = 6$ ft., so that δ will be $6 \times 1.27 = 7.62$ ft. This section is shown at (a) in Fig. 19. The total capacity will be 168

cub. ft. per foot run, and the area of sheeting $12 + 22.97 = 34.97$ sq. ft. per foot run. A semi-cylindrical trough to give the same capacity would be 20.69 ft. in diameter, and its area of sheeting 32.51 sq. ft. per foot run. Here the increase is 2.46 in 32.51, or about 7.6 per cent. The equivalent semi-circle is shown in the sketch for comparison.

Second, let $d = 7$ ft., so that δ will be $7 \times 1.27 = 8.89$ ft. This section is shown at (b) in Fig. 19. The total capacity will be 196 cub. ft. per foot run, and the area of sheeting $14 + 24.97 = 38.97$ sq. ft. per foot run. A semi-cylindrical trough to give the same

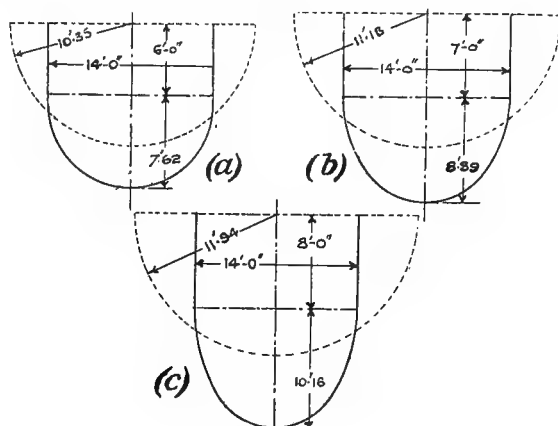


FIG. 19.

capacity would be 22.35 ft. in diameter, and its area of sheeting 35.14 sq. ft. per foot run. The increase in this case is 3.83 in 35.14, or about 10.9 per cent.

Third, let $d = 8$ ft., so that δ will be $8 \times 1.27 = 10.16$ ft., as at (c) in Fig. 19. The total capacity will be 224 cub. ft. per foot run, and the area of sheeting $16 + 26.97 = 42.97$ sq. ft. per foot run. A semi-cylindrical trough to give the same capacity would be 23.88 ft. in diameter, and its area of sheeting 37.5 sq. ft. per foot run. Here the increase is 5.47 in 37.5, or about 14.58 per cent.

These increases are not all loss, however, for the semi-cylindrical trough taken as the standard of comparison would require supporting work in addition, probably costing much more than would the

increases in sheeting area required for the more convenient rectangular tanks with trough bottoms.

It would be the reverse of advantageous to put more of the capacity into the troughs for these cases, unless the breadth could be increased. A little consideration will show that, keeping the breadth unaltered, the trough would become deeper if its volume were increased, and the divergence from the equivalent semi-circular cross-section would be greater. Saving might be effected with the largest of the three if the capacity of the trough were made double that of the rectangular portion, the breadth being increased to 18 ft.

The depths would then be : $d = \frac{224}{3 \times 18} = 4.15$ ft., and $\delta = 4.15 \times 2 \times 1.27 = 10.54$ ft., giving the area of sheeting $8.3 + 30.71 = 39$ sq. ft. per foot run. This section is shown in Fig. 20. A semi-cylindrical trough to give the same capacity would be 23.88 ft. in diameter, and its area of sheeting 37.5 sq. ft. per foot run, so that the increase for the more convenient form would be only 1.5 in 37.5, or about 4 per cent.—a very considerable reduction from the 14.58 per cent. for the same capacity with the width 14 ft. and the trough equal in capacity to the rectangular portion.

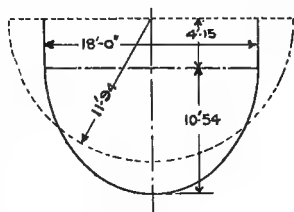


FIG. 20.

These results show clearly the advantage, as regards area of sheeting, of putting a large proportion of the capacity into the trough provided the breadth be sufficient to give a fair degree of approximation to the equivalent semi-circular shape.

For the shorter rectangular tank with a trough bottom—*i. e.* where the sheeting in the ends is *not* insignificant as compared with that in the sides—mathematical investigation is unnecessary. From the facts it follows obviously that the form giving greatest economy of sheeting area would be a hemispherical bowl, and the endeavour, in selecting proportions for a particular case, should be to depart as little as may be from the equivalent hemispherical bowl.

26. *General Considerations regarding Shaped Bottoms.*—A few general points regarding curved or dished bottoms may be noticed in concluding this consideration as to economy of form.

It has been shown that the curved form is economical as regards sheeting area, and also as regards support and stiffening. On the other hand, the costs of manufacture, transport, and erection are inevitably greater with curved than with flat sheeting. It is, therefore, quite possible (and has frequently happened) that a tank having a dished bottom may be lighter, and yet cost considerably more than would the plain flat-bottomed tank, with its supporting joists, to give equal storage capacity.

No hard-and-fast rule can be laid down, of course, for circumstances of locality and available labour must very largely influence particular cases, but it is probable that, for cylindrical tanks, flat bottoms with supporting joists are cheaper than dished bottoms for diameters less than 20 ft. In any case, economical manufacture for dished bottoms cannot be expected unless the diameter and shape have been adopted as stock, dies and forms for the shaping being already available. The plates for such a bottom cannot be properly shaped without forms, and even then they must be worked hot. Hence, if the diameter of the tank be selected without reference to forms which manufacturers may have available, it will be necessary to either make new forms or adapt existing ones, and the cost (charged, of course, to the one job in full) will be great, if not prohibitive.

For rectangular tanks with trough bottoms, the advantages of the trough become effective for smaller widths than with cylindrical tanks, because the bending and other work in connection with the plates for the trough is less costly than for the spherical or spheroidal dish. Probably the flat bottom with supporting joists will cease to be more economical in cost where the width is upwards of 12 ft. or 15 ft.

Always, circumstances must influence the form selected. In perhaps the majority of cases in ordinary practice, economy in the tank itself is a secondary consideration, giving way to the demand for convenience or space utilisation, and then the best that can be done is to obtain the most economical design for a specified form and (more or less) imposed set of proportions.

The matter of curved bottoms for tanks—in this country at least—is somewhat difficult from the commercial point of view. There has not been sufficient demand for such work to induce manu-

facturers generally to adopt stock sizes and proportions and make the necessary forms and dies. On the other hand, so long as forms are not available, any tendency to create a demand is checked by the high costs of manufacture, and until these opposing considerations are reconciled it will be impossible to secure the full advantages possessed by dished and trough bottoms.

An attitude which designers might well adopt is a constant readiness to use dished or trough bottoms for tanks where advantage may be obtained by so doing, coupled with an insistent and proportionate demand for conclusive evidence as to the reality of the advantage. This, of course, implies an intimate and accurate knowledge on the part of the designer, as to the costs of manufacture, transport, and erection of all classes of work, and the ability to judge correctly as to which methods are likely to be most economical in each individual case. It implies also an impartial view of all available methods, and not a hidebound preference for one form solely because it is easy to design, and without reference to its merits, both actual and relative. It might be well if designers were to consult manufacturers more than they do, before deciding on a particular form for a tank, so as to ascertain whether the work involved in a tentative design can be done at reasonable cost, or (as sometimes happens, and is undesirable from almost every point of view) is the exclusive speciality of one yard.

We shall turn next to the design of tanks generally, as regards strength and stiffness, arrangement of plates, connections, methods of erection, and such matters. In later chapters, dished and trough bottoms are considered from these points of view.

CHAPTER III

FLOORS AND WALLS OF RECTANGULAR TANKS

27. *Tank Floors and Walls Generally.*—The loading to which the walls and bottom of a tank are subjected is, in general, that due to the weight of the contained liquid. It is not often that the liquid is stored under pressure; but even if it were, the ordinary gravitational loading would simply be increased by the amount of pressure imposed.

It is commonly stated that the pressure per unit area at any level in a liquid acts in all directions, and is equal to the weight of the column of liquid which would stand upon the unit area, of height equal to the vertical distance between the upper surface of the liquid and the level in question. This is strictly true where the area under pressure is horizontal, but for a vertical or inclined surface it involves the contradiction that, since the intensity of pressure varies directly with the depth, the pressure cannot be uniform over a whole unit of area. For the purposes of practical design, however, it is sufficiently accurate to work as though the pressure intensity were constant over the whole area under consideration, its magnitude being taken as the greatest likely to act.

Obviously, to design the walls of a tank thus causes a waste of material, for, since the loading varies with the depth, so also, to secure economy of material, should the strength of the sheeting. Plates of diminishing thickness are not easily obtainable, however, and would be costly to produce. The diminution in thickness for the wall of a rectangular tank would need to be parabolic to give uniform diminution in strength, since the resistance to bending of a plate varies as the square of its thickness.

Where the tank is of considerable depth, saving may be effected by making the plates of each strake thinner than those of the strake immediately below, but the strakes are usually of the maximum

width convenient to use. Each strake must be riveted to its neighbours, and the seams must be tight against leakage. Every strake used, then (after the first), means a horizontal seam all round the tank, and, as a rule, it is cheaper to avoid this riveting as much as possible, even at the expense of sheeting. Doubtless, if good riveting were less troublesome and costly to obtain, tanks might be designed more economically, in some cases, by the use of narrower strakes, but under ordinary circumstances in this country the main object is to minimise riveting. It must be remembered, also, that riveted seams take time to make and caulk properly, especially if the work must be done during erection at the site. For all ordinary widths, the price of mild-steel plates per ton is the same, and hence, against the saving which might be effected through frequent reductions in the thickness of the plating with narrow strakes, there must be set not only the cost of the riveted seams, but also the cost of the extra time required to make them.

We will consider first the design of the tank bottom, returning to the side walls afterwards.

28. *Design of the Rectangular Flat Floors.*—Where the tank bottom rests directly upon a solid base of concrete or other similar material, there is, of course, no need to design the plates for strength. The minimum thickness allowable (generally $\frac{1}{4}$ in.) may then be used, and the riveting sufficient only to secure tightness of the seams against leakage.

For a flat bottom supported on bearer-joists, the plating is, clearly, in the position of a beam, uniformly loaded. When calculating to find the thickness required, it is convenient to consider a strip of the plate 1 ft. in width, since all such strips are similarly loaded.

Conflicting views are held and expressed as to what should be taken for the span of this beam. Some say the distance between the centres of the supporting joists; others the clear distance between the flanges of those joists; and others distances intermediate between these two. Others, again, contend that the plates will act as a continuous beam, with points of contraflexure between the supporting joists; and that the distance between these points of contraflexure should be taken as the span.

In fact, the question depends largely upon circumstances. If

the bottom were formed of a single plate, perfectly flat, and the supporting joists very narrow and stiff, with perfect bearing of the plates throughout, the conditions would be those of the continuous beam. The points of maximum bending moment would then be over the supporting joists, however, and if the pitch of the joists were constant throughout the length of the tank, with a joist under each end wall, the bending moment in the plate at the joists next to those at the extreme ends would not be less than that for a freely supported beam-strip spanning from joist to joist. Hence, there would be no saving, for the plate must be throughout of the thickness required for the position of the maximum bending action. Of course, the end spans, and those next to them, could be made less than those towards the middle of the tank, and the bending moments thus kept more nearly equal throughout the length of the bottom; but it will be clear that trouble and expense would be involved, reducing the advantage to be obtained.

In actual tank work the conditions are (the author considers) not such that reliance could be placed upon the plates acting in accordance with the theory of continuous beams, though, of course, since the plates are continuous over the supports, there must be some such action. As the extent of the action cannot be estimated with any reasonable probability of truth, it is better that it be not calculated upon to any great extent.

The argument for taking the span as the full distance between the centres of the bearer-joists is based upon the assumption that the flanges of the joists will bend easily when the load comes upon them, leaving the plate supported over the joist webs. This may be regarded as an unnecessarily severe assumption, though, of course, there is not much difference between the distance between the centres and that clear between the edges of the flanges in ordinary cases.

A little consideration will show that, having regard to the circumstances of material and workmanship in such structures, no definite theory of action can be laid down. The best that can be done, for the purposes of practical design, is to obtain a simple and convenient relation which may be reasonably depended upon to give safe results without avoidable waste of material.

With regard to this question of necessary and minimum per-

missible plate-thicknesses, it should always be remembered that the use of a slightly greater thickness than that of the absolutely irreducible minimum has the advantage of giving a longer life to the structure, as well as tending to reduce the liability to leakage through opening of the seams. There can be no justification for useless piling up of weight, but where a calculated thickness lies between two stock thicknesses, and the question arises whether the next below or the next above shall be used, it should not be forgotten that the advantage of the "cut" thickness is confined to first cost alone, whereas the stouter plate will most likely give better value in a greater degree of permanence.

The method here suggested for design is to space the bearer-joists equally, leaving about one-third spaces overhanging at each end, as indicated in Fig. 21, and to design the plate as though its

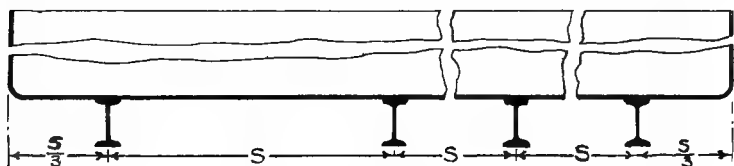


FIG. 21.

span, both for loading and supporting, were the clear distance between the edges of the bearer-joist flanges, with ends freely supported. This, of course, is a compromise, for the actual conditions are probably not in accordance with the assumptions made; but the thicknesses so found have proved reliable in practice.

A simple expression for the thickness of the plate may be deduced on this basis, as follows—

Taking the span L and the depth D (Fig. 22), both in feet, the load on a strip of plate 1 ft. in width will be $(L \times D \times 1)$ cub. ft., at 62.5 lb. per cub. ft. if the liquid be water. Then, the load on the beam-strip of plate will be—

$$W = (62.5 \times L \times D) \text{ lb.} = \frac{62.5 \times L \times D}{2240} \text{ tons};$$

and the maximum bending moment will be—

$$B = \frac{12}{8} \frac{W L}{8} = \frac{12 \times 62.5 L^2 D}{8 \times 2240} = \frac{75 L^2 D}{1792} \text{ in.-tons.}$$

Then, for a depth of 5 ft.,

$$t = \frac{2.75 \times \sqrt{5}}{20} = \frac{11 \times 2.236}{4 \times 20} = \frac{5.09}{16},$$

and the thickness should be $\frac{3}{8}$ in. The same thickness will be sufficient for depths of 6 ft. and 7 ft., the comparatively small margin for the latter depth taking the thickness to $\frac{7}{16}$ in. if any allowance be necessary to provide for corrosion. For a depth of 8 ft. or 9 ft., the thickness should be $\frac{7}{16}$ in., and for depths of 11 ft. or 12 ft., the thickness should be $\frac{1}{2}$ in. This, of course, is on the

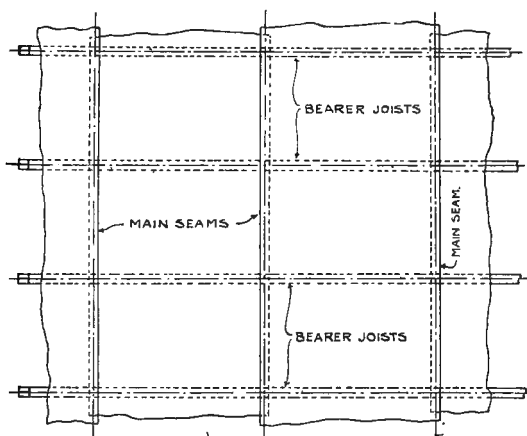


FIG. 23.

assumption that the plates hold the nominal thickness with reasonable uniformity throughout. If there be any reason to suppose that the actual thickness may be less in parts than the nominal thickness, a proper and corresponding allowance should be made in the thickness selected.

To the thicknesses obtained from equation (32) must be added any allowance necessary to provide for corrosion, but in most cases—at least for plates $\frac{1}{2}$ in. and over in thickness—the allowance for corrosion may start from the thickness obtained as the value of t from the equation. Thus, if the additional thickness to allow for corrosion is to be $\frac{1}{32}$ in., with $L = 2.75$ and $D = 8$ ft., the value

of t as found from the equation will be $\frac{6.226}{16}$, which is slightly under $6\frac{1}{4}$ sixteenths, and adding the $\frac{1}{32}$ in. for corrosion, the thickness is still about $\frac{1}{84}$ in. less than $\frac{7}{16}$ in. The plate might, then, be made $\frac{7}{16}$ in. in thickness, for all but exceptional circumstances. Of course, all such points must be dealt with according to the particular circumstances and conditions of each individual case, a selection of thickness perfectly reasonable in one case being unjustifiable in another.

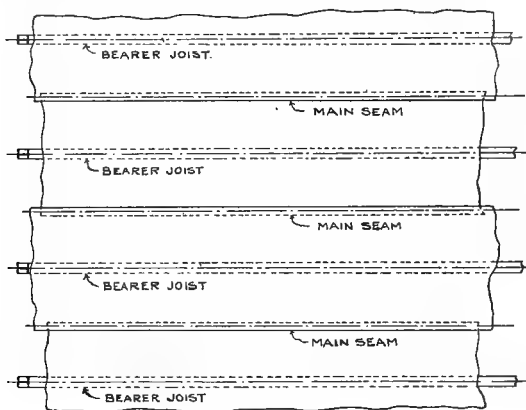


FIG. 24.

29. *Seams in Flat Floors.*—Where the length and breadth of the tank are such that seams are unavoidable, the plates should be so arranged that the seams will lie transversely to the bearer joists, as indicated in Fig. 23. By this means, the disturbance to the beam action of the plates, and hence the tendency to leakage through the opening of the joints, will be less than with the seams running parallel with the joists, as in Fig. 24.

The rivet heads should be countersunk on the underside where the seams cross the supporting joists, so that the bearing may be as even as possible.

The seams now under consideration are those connecting plates which run in one piece from end to end, or from side to side (whichever may be the most convenient), of the tank bottom. They may

be conveniently termed "main" seams, in contradistinction from the "subsidiary" seams which are necessary when the length of the strips is so great that a single plate would be impossible or impracticable.

It is open to question whether lap or butt joints should be used for the main seams. The consensus of expressed opinion is in favour of the single-riveted lap joint, solely on the score of cheapness.

Since tightness against leakage is the only condition to be satisfied in such seams, single riveting is sufficient for either type of joint, and from this it follows that the butt joint demands two rivets for every one required by the lap joint, while the butt with a single cover strip involves twice, and that with double cover strips three times, as many rivet holes as are necessary with the lap joint.

There need be no extra edge-planing, however, with the butt joint, for the cover strips may be of stock flat bars, the edges of which seldom require planing to render them suitable for caulking. Even where lap joints are used, unless some special means are adopted at the mills to secure fair edges, it is only in the roughest and poorest work that the edges are allowed to go unplanned, for the relatively small saving effected by omitting the planing of bad edges is nearly always lost many times over, through the increased difficulty in caulking to render the seams tight, and the time thus wasted. There is, of course, more caulking (as regards length) to be done with butt joints than with lap joints, but, on the other hand, the caulking is much easier to do with the former, and more likely to be satisfactory, as the plates are better supported.

A point in favour of butt joints for the main seams lies in the fact that they give the tank a flat bottom, and, consequently, a better bearing on the supporting joists. If single cover strips are used, they may be placed on the inside, and if double cover strips be preferred, those on the underside may be stopped just clear of the flanges of the supporting joists.

With lap joints, the best arrangement is that indicated in Fig. 25, with packing strips to give the alternate plates a bearing upon the supporting joists. These packing strips should have the same width as, and be tacked to, the joist flanges. Two rivets to each strip are sufficient if the strips be reasonably straight and flat. If

the bearing joists are to act as continuous beams (which is the most economical plan), there will be the least possible reduction in the strength of the joists if the tacking rivets be placed one at each extremity of the middle half of the span—*i. e.* at a quarter of the span from each beam carrying the bearer joists,—for that is the neighbourhood of the points of contraflexure in the joist; or, of course, the tacking rivets will do no harm if they lie well within the middle half of such span, for there the upper flange of the bearer joist will be in compression. If the supporting joists are to act over separate and freely supported spans, the tacking

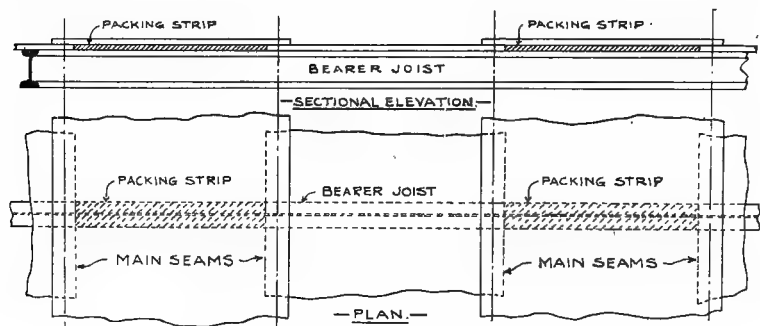


FIG. 25.

rivets should be placed as near the ends of those spans as is practicable. The heads of the tacking rivets must be countersunk at the upper sides of the packing strips, of course.

For shallow tanks, where the thinnest practicable plates ($\frac{1}{4}$ in. being usually the minimum permissible thickness) for the bottom are stressed well below the allowable working stress, the packing strips of Fig. 25 may be dispensed with by altering the arrangement of the plates and seams. If the main seams lie parallel with the supporting joists, and the distance between adjacent seams be 50 per cent. more than the pitch of the joists, the arrangement of Fig. 26 will cause the seams to be placed at or near points of contraflexure in the bottom-plating. The joists which are to carry the raised plates of the bottom should be suitably packed from the main or secondary beams, as shown, the packing pieces being of the area comprised in the intersection of the two flanges, and of thickness equal to that of the bottom-plates.

All things considered, and given the same class of work and standard of requirements for both, it is probable that lap joints for the main seams are cheaper than butt joints, but the difference in cost is not nearly so much as might appear from a cursory examination. Moreover, in cases where a thoroughly sound job is regarded as of more consequence than a perhaps trifling reduction in first

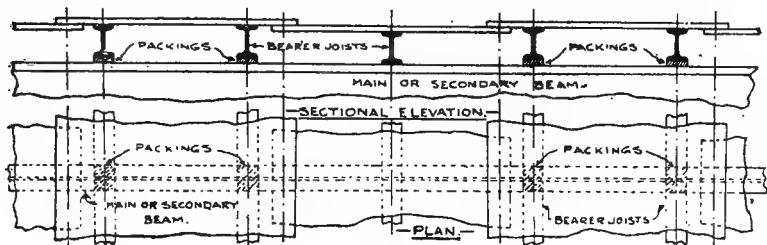


FIG. 26.

cost, and where the need for rapidity in installation demands that the time absorbed in trials and re-caulkings before the tank is ready for service shall be minimised, it is probable that the use of butt joints for the main seams can be justified from all points of view.

When subsidiary seams in the bottom-plates are unavoidable, the object should be to place them where there is the least probability of their being opened so far as to permit leakage. With the arrangement of Fig. 23, such seams must, of course, lie perpendicular to the main seams, and, hence, parallel with the supporting joists. To subject a joint, whether of the lap or butt type, to a severe cross-bending action, as indicated in Fig. 27, must, obviously, tend to open the joint, and as the strength of such a joint to resist cross-bending would be practically impossible to estimate with any reasonable probability of truth, the endeavour should be to prevent the application of such actions.

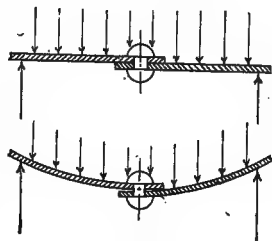


FIG. 27.

It seems reasonable to suppose that if a subsidiary seam be

placed at about one-fourth of the distance between the joist-centres from a supporting joist, as indicated in Fig. 28, the bending action upon it cannot be very severe, because of the effects of continuity.

Subsidiary seams, it is here suggested, should always be butt joints, with double cover strips. Besides giving a seam less likely to open, such a course is, as will be seen presently, of assistance in securing a convenient arrangement of the plating.

Another method of dealing with the subsidiary seams is to stiffen the bottom-plates in the neighbourhood of such seams. The seam may be placed midway between the supporting joists, and three

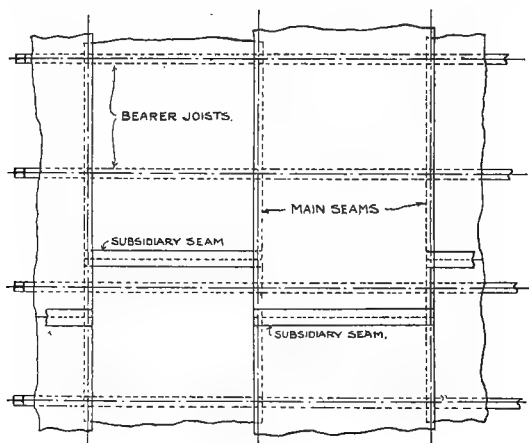


FIG. 28.

stiffeners, of fairly stout angle section—say, $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in.—riveted to the bottom-plates, as shown in Fig. 29. Two of the angle stiffeners may be incorporated in the main seams at each side, and the third one placed across the centre of the subsidiary seam. It is best to attach these angles to the underside of the bottom-plates, thus allowing the inner cover strip of the subsidiary seam to pass from main seam to main seam, and to be well caulked, while the outer cover strip may be cut to fit between the angles and main seams, as shown in the sketch. The rivets securing the middle angle stiffener to the bottom-plates may be $\frac{5}{8}$ in. or $\frac{3}{4}$ in. diameter and 6 in. pitch.

Other methods will doubtless suggest themselves, and it is only necessary to remark that anything in the nature of a trimmer lying *along* the subsidiary seam is undesirable, since the effects of continuity in the plating might cause the most severe bending action to be applied over such a secondary support—*i. e.* in the seam itself—and thus defeat the very object in view.

30. *Riveting in Floor Seams.*—Where lap joints are used throughout the tank bottom, a difficulty arises at the junction of a subsidiary with a main seam, owing to the interference of the third plate. If the plates were brought together without preparation for this difficulty, the conditions would be as indicated in Fig. 30—

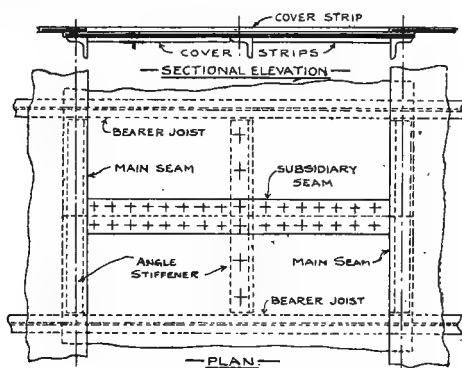


FIG. 29.

two of the plates either butting, as at (a); or separated by a space equal to the thickness of the middle plate, as at (b).

The best known (and most widely adopted) method for overcoming this difficulty consists of arranging the plates as at (a) in Fig. 30, thinning out the corner of the middle plate to a feather edge, and bending that of the newcomer, as shown in Fig. 31. Provided the work be well done, this method gives a satisfactory job. It is, however, somewhat costly, for the two plate corners must be worked by forging, and care is necessary to ensure that all the surfaces in contact shall lie truly flat, since any spring would be fatal to tightness of the joint against leakage. Plates of even comparatively small size are awkward to hold and handle, both for heating and working, and consequently the cost is high. Very

careful caulking is required with this method, especially around the feathered edge of the thinned plate.

Another method, indicated in Fig. 32, consists of bending the third plate only, and introducing a tapered packing-piece at the end of the middle plate. This reduces the number of plates to be forged, but the smithing on those which must be forged is more complicated, and, as the tapered packing-pieces need to be carefully prepared in addition, it is probable that an impartial investigation as to costs would show little (if any) advantage for this method as compared with that of Fig. 31. Even more care in caulking is

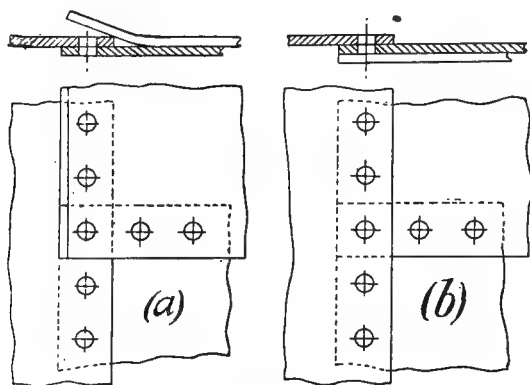


FIG. 30.

required here, for there is the additional straight joint where the packing butts against the edge of the middle plate.

The author is strongly of opinion that the best method is to use a butt joint for the subsidiary seams, as suggested in pp. 76-81. A little more weight is involved, and a few more rivets and holes, but there is no forging of plate corners required. For a tank of reasonable magnitude, and in ordinarily good-class work, it is probable that, on the question of actual costs for labour and material directly involved, this method would compare very favourably with either of those described above, while it would almost certainly give a considerable saving in time at the site, for the processes of assembling and caulking would unquestionably be simplified.

Yet another method is available. The plates may be arranged

as at (b) in Fig. 30, with a space between the extremes, and this space may be filled with a plain, straight packing strip, of width equal to the lap of the plates for the seam. The arrangement is shown in Fig. 33, and if the treatment be confined to the subsidiary seams, the method may prove both satisfactory and cheap. As will be seen, it permits the use of single-riveted lap joints throughout. Two disadvantages—perhaps not very serious ones—of this method are: (1) The increased irregularity of the tank bottom, requiring more packings to give the plates a working seat upon the supporting

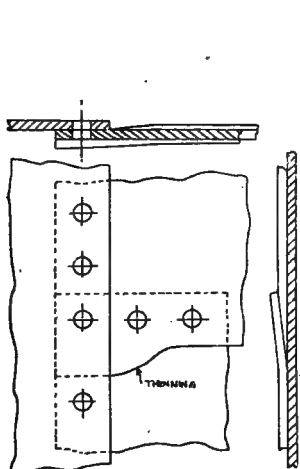


FIG. 31.

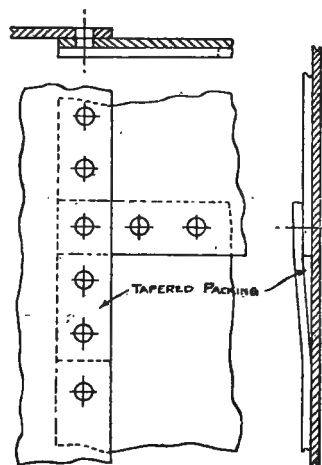


FIG. 32.

joists; and (2) the increased tendency towards weakness in the seam for resisting the cross-bending actions which may be applied to it.

With regard to the question of economical spacing for the supporting joists, in so far as it affects the thickness of the bottom plates, it will be clear that there are two opposing considerations. For economy in the plates of the bottom, it is desirable that the supporting joists be as close together as possible; and for economy in the joists themselves, they should, obviously, be as far apart as possible. The question is evidently affected also by the spacing of the beams which carry the supporting joists, for that determines the span of those joists.

31. *Attachment of Walls to Rectangular Tank Floors.*—There are two principal methods in more or less common use for connecting the wall plates to the floors of rectangular tanks. One is by means of angle bars, as indicated at (a) in Fig. 34; and the other is by flanging the bottom plates, as at (b) in the same sketch.

The former method becomes troublesome if the main seams are

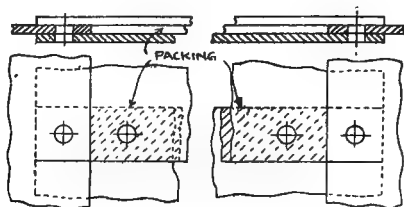


FIG. 33.

formed of lap joints, for there is a space between the angle and alternate plates, as shown in Fig. 35. Packing strips might be used to fill these spaces, but such devices are often regarded unfavourably, and consequently the angle bar connection is used mostly

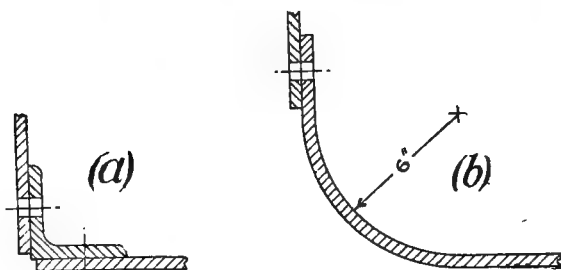


FIG. 34.

where the plates are thin, and can easily be drawn down at the corners to fit closely against the angles.

Clearly, if the seams in the walls and bottom be of the butt type, the angle bar connection becomes very simple and convenient, for the plates will all lie flat against the angle flanges, and the inner cover strips may be stopped close against the edges of the angles.

The second method—as at (b) in Fig. 34—is frequently used.

By means of suitable dies and forms, the flanging of the plates may be done well and cheaply, though the plates must, of course, be properly heated first. This method, also, becomes very simple and convenient if all seams in the walls and floor of the tank be of the butt type. With lap joints it is necessary to provide for the interference of the third plate at the junctions of seams, and this is usually done by thinning down the corner of the middle plate.

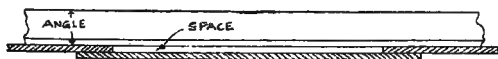


FIG. 35.

The seams in the walls should be placed midway between those in the floor, and the arrangement will then be as shown in Fig. 36.

Here, again, it is probable that the better method is to use packing strips for the alternate spaces so as to avoid the heating and forging of the plates at the corners otherwise necessary; thin-

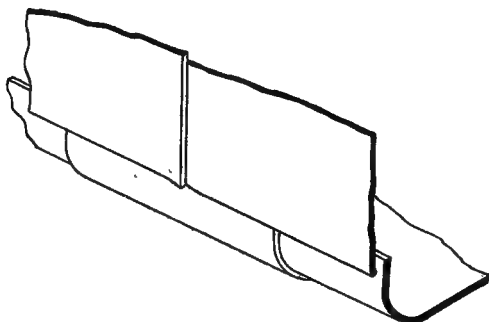


FIG. 36.

ning down the flanged corner of a bottom plate is particularly troublesome, and a troublesome operation is, almost invariably, costly also.

With the second method the plates at the ends of the bottom must be flanged along two edges, and the corner made spherical, as indicated in Fig. 37. Also, the plates forming the corners of the walls must be bent to connect with the bottom, as shown. This needs to be very carefully done, and requires specially suitable appliances; but, on the other hand, such a corner needs little

caulking, and this saving should be taken into account when comparing the cost with that of a corner connection formed by means of an angle bar.

To minimise the necessary forms and dies for flanging and bending the plates, it is well to use a stock radius—about 6 in. is convenient for most cases—for all tanks, regardless of size and plate thicknesses. Provision should be made for the difference in radius where one plate lies inside or outside another, and this may be done by inserting packing strips between the dies and forms. A complete set of such packings should be kept, and regarded as part of the flanging apparatus. They may be made

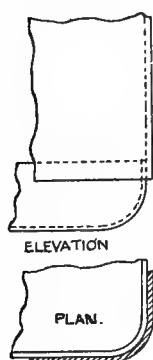


FIG. 37.

from strips of ordinary stock plates, of suitable widths and lengths, and in all the usual working thicknesses. The dies and forms must be made with sufficient difference in radius to accommodate the packing strips, of course. This may seem rather in the nature of a refinement; but it will be found that a little trouble and care taken in preparation, to ensure that the pieces shall all come together properly when assembled, is nearly always a sound investment.

32. *Cantilevered Walls of Rectangular Tanks.*—If the side walls of a rectangular tank were not stayed or braced in any way, they would be, in effect, simple retaining walls, and would have to possess, through their own mass, adequate resistance to movement and overturning. There are, of course, tanks—or, at least, structures which fulfil the purposes of tanks—for which the side walls can only be designed and constructed on this basis, but such types do not lie within the scope of this work. Modifications of the retaining-wall principle may, however, be applied sometimes with advantage, as will be shown presently.

For tanks constructed of steel sheeting, or of such materials as reinforced concrete, the side walls almost invariably require to be stayed or braced in some way; and as the design of the sheeting or panelling depends to a large extent upon the disposition and effects of the staying, it will be well to consider the various ways in which the walls may be stayed, having regard to their influences upon the economy of sheeting or panelling.

Before proceeding to this, however, we may consider one case in which no special bracing (*i. e.* no additional ties or framing) need be provided for supporting the side walls, the only requisite being that the walls shall be connected to the floor with adequate strength, stability, and stiffness. This case is the shallow tank which forms the dished floor and collector for an elevated cooling tower, such as is used for cooling the circulating water for the condensers working in connection with steam turbines. Instances of this type of tank occur also in other circumstances where large horizontal area is either unavoidable or more desirable than strict economy of form.

Such tanks are usually about 2 ft. to 3 ft. in depth, and may be conveniently constructed of steel plates. A suitable and common arrangement for the plating is indicated in Fig. 38, the plates being preferably connected by means of single-riveted lap joints for the main seams, and single-riveted, double-covered butt joints for the subsidiary seams. If a curb is required around the brim of the tank, it may be of light angle section, with flat bar packings to the alternate plates, which will be recessed by reason of the lap seams. The riveting for the curb need be only so much as is required for the purpose, and it is clear that tightness against leakage can scarcely be an important condition for such riveting.

In such tanks as that illustrated in Fig. 38, the side walls stand as cantilevers, and the stresses in the plates should be estimated on this basis. Since each section of the side walls is in one piece with the corresponding bottom plate, it follows that there is a maximum permissible depth or head of the contained liquid for each thickness of plate if the maximum permissible working stress in the material is to hold as far as possible throughout the structure.

We will investigate the conditions as to the effective length and loading of the cantilever portions, and determine the maximum allowable heads for each plate thickness in common use.

Clearly, the conditions for the end walls will be somewhat different from those for the side walls, since the bearer joists lie transversely to the former and parallel with the latter. We shall return to this point presently, and examine the difference and its practical effects upon the design.

Let us consider the end walls, with the bearer joists running

transversely. An enlarged section of the parts concerned is shown for clearness in Fig. 39, with the bearer joists indicated by dotted lines. For simplicity in the investigation, we will confine our attention to a strip of the wall 1 ft. in length.

The maximum bending moment will occur at the section A A,

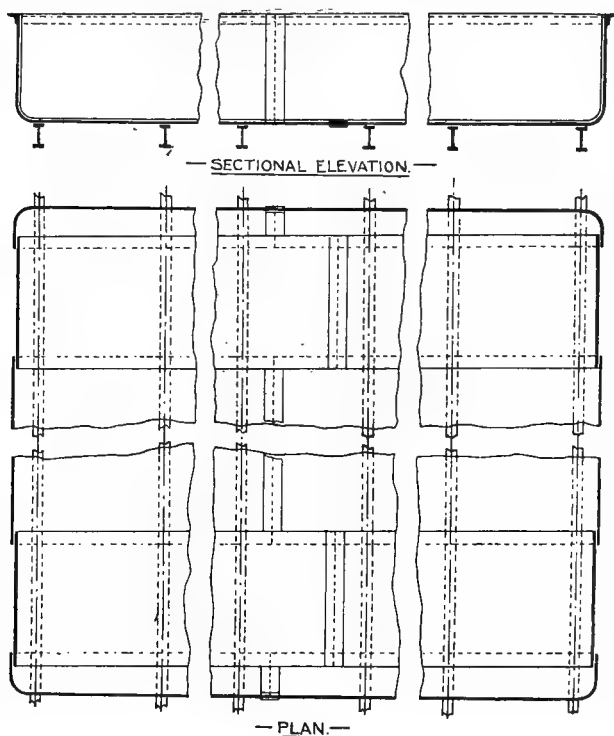


FIG. 38.

where the bottom plates lose the support of the bearer joists, and the arm of the cantilever should be taken, rather full than short, as the distance (indicated as L in Fig. 39) between the section A A and the surface of the liquid, measured along the plate.

To determine precisely the variations of loading and stress for the cantilever strip under a given head of the contained liquid might be interesting as an exercise, but would occupy more time than could usually be spared in commercial offices. The diagram of

loading would be somewhat as indicated at (a) in Fig. 40, and the correct load diagram for a given head might easily be plotted; but the bending moment would obviously be troublesome to calculate. Moreover, since the head and density of the contained liquid are neither constant nor accurately known, and the plates must be in stock thicknesses, no useful purpose would be served by employing such refinements in calculation. It is much easier, and quite as reliable for practical purposes, to take the effective head as equal to the cantilever arm, and to work on this as though the intensity of loading were simply proportional to the distance along

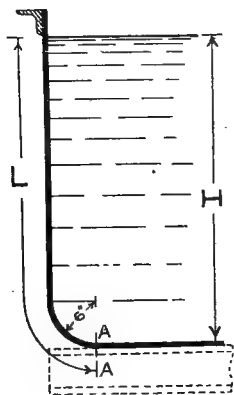


FIG. 39.

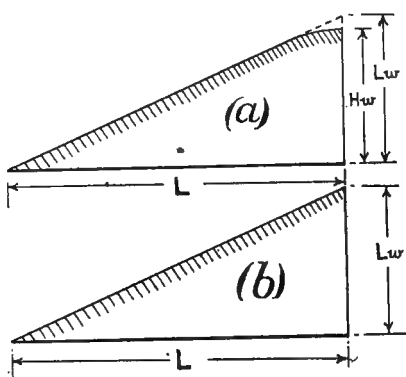


FIG. 40.

the cantilever arm measured from the surface of the liquid, as indicated at (b) in Fig. 40.

If the arm of the cantilever be represented by L (measured in feet), the average intensity of loading will be $\frac{Lw}{2}$ pounds per square foot, where w is the weight of the liquid in pounds per cubic foot. For water, w may be taken as 62 lb., giving the average intensity of pressure as $31 (L)$ lb. per sq. ft. The total load on the cantilever strip will be $31 (L^2)$ lb., and the resultant pressure will act at a leverage of $\frac{1}{3} (12 L)$ in. = $4 L$ in. from the section A A. Hence, the bending moment at A A will be—

$$\begin{aligned} B &= 31 (L^2) \times 4 (L) = 124 (L^3) \text{ in.-lb.} \\ &= \frac{124}{2240} (L^3) = \frac{31}{560} (L^3) \text{ in.-tons.} \end{aligned}$$

If t represents the plate thickness in inches, the section modulus of the strip will be—

$$M = \frac{1}{6} (12 t^2) = 2 t^2 \text{ in.}^3\text{-units};$$

and with a permissible maximum stress of 7.5 tons per sq. in. in the material, the resistance moment will be—

$$R = 7.5 \times 2 t^2 = 15 t^2 \text{ in.}^3\text{-tons.}$$

Hence—

$$\frac{31}{560} (L^3) = 15 t^2; \text{ or, } L^3 = \frac{15 \times 560}{31} (t^2) = 271 t^2,$$

and

$$L = \{\sqrt[3]{271 (t^2)}\} = 6.4713 \{\sqrt[3]{(t^2)}\} \text{ ft.}$$

Inserting appropriate stock values for t , the following table may be constructed:—

t = Thickness of Plate in Inches.	Calculations.	L = Length of Cantilever Arm, in Feet.	H = Maximum Permissible Head in Feet.
$\frac{1}{4}$	$L = 6.4713 \left(\sqrt[3]{\frac{1}{16}} \right) = \frac{6.4713}{2.5198}$	2.57	2.28
$\frac{5}{16}$	$L = 6.4713 \left(\sqrt[3]{\frac{25}{256}} \right) = \frac{6.4713 \times 2.9240}{6.3496}$	2.98	2.69
$\frac{3}{8}$	$L = 6.4713 \left(\sqrt[3]{\frac{9}{64}} \right) = \frac{6.4713 \times 2.0801}{4}$	3.36	3.07
$\frac{7}{16}$	$L = 6.4713 \left(\sqrt[3]{\frac{49}{256}} \right) = \frac{6.4713 \times 3.6593}{6.3496}$	3.73	3.44
$\frac{1}{2}$	$L = 6.4713 \left(\sqrt[3]{\frac{1}{4}} \right) = \frac{6.4713}{1.5874}$	4.08	3.79
$\frac{9}{16}$	$L = 6.4713 \left(\sqrt[3]{\frac{81}{256}} \right) = \frac{6.4713 \times 4.3267}{6.3496}$	4.41	4.12
$\frac{5}{8}$	$L = 6.4713 \left(\sqrt[3]{\frac{25}{64}} \right) = \frac{6.4713 \times 2.9240}{4}$	4.73	4.44
$1\frac{1}{8}$	$L = 6.4713 \left(\sqrt[3]{\frac{121}{256}} \right) = \frac{6.4713 \times 4.9461}{6.3496}$	5.04	4.75
$\frac{3}{4}$	$L = 6.4713 \left(\sqrt[3]{\frac{9}{16}} \right) = \frac{6.4713 \times 2.0801}{2.5198}$	5.35	5.06

In determining the maximum permissible depth or head, it must be remembered that L is to be measured along the flanged plate.

If the flanging is 6 in. radius, as shown, the quadrant length will be $\frac{2 \times 6 \times 3.1416}{4} = 9.42$ in. = 0.79 ft., so that, as the arm of the cantilever strip gains 0.79 ft. while the actual depth gains only 0.5 ft., the calculated values for L should be reduced by 0.29 ft. (*i. e.* the excess of 0.79 ft. over 0.5 ft.) to give the maximum permissible heads (H in Fig. 39) as in the last column of the accompanying table.

Returning to the side walls, lying parallel with the bearer joists, it will be clear that the bending moment in the plating depends upon the spacing of the bearer joists. The object, for all-round economy and suitability, should be to so arrange that the maximum bending moment from the cantilever portion occurs at the section where the bottom plate leaves the horizontal and commences its upward curve, since that is the condition in the end walls.

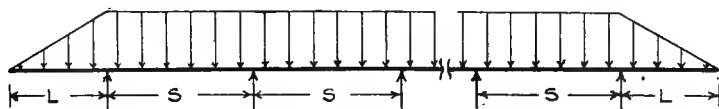


FIG. 41.

Although, as has been shown, it is not much use to apply elaborate continuous girder theory in the design of tank floors, some degree of approximation to the truth may certainly be obtained by taking account of the reasonable probabilities. In this case, if the side walls were turned down into the plane of the floor, and loading applied throughout to produce everywhere effects similar to those of the contained liquid in the tank, the conditions would be as indicated in Fig. 41. Then, regarding the plate as a continuous beam, and assuming constant moment of inertia, with all supports at one level and of uniform rigidity, the stresses could be determined.

The bending moment will attain maximum values at those sections where the slope of the plate-beam is zero, and it is clear that the most economical arrangement will be that which gives zero slope at all supports, including the extremes. This condition corresponds to a bending moment of $\frac{WS}{12}$ at each support, W

being the total load on each span, and S the length of each span. Considering a strip 1 ft. in width, as before, H and S being measured in feet, we have $W = w H S = 62 H S$ for water. The bending moment at each support will be—

$$B = \frac{62 H S \times 12 S}{12} = 62 H S^2 \text{ in.-lb.}$$

Equating this with the bending moment from the cantilever strip—

$$62 H S^2 = 124 L^3;$$

and if H be regarded as sensibly equal (for the present purpose) to L —

$$S^2 = 2 L^2, \text{ whence } S = 1.414 L.$$

To allow for the fact that L is, in reality, slightly more than H , and for convenience in design, we may take $S = 1.5 L$.

A design based on these proportions will be dependably sound provided that care be taken to ensure that the assumptions shall be fulfilled so far as may be reasonably practicable. The bearer joists must be sensibly straight, and of uniform section; the plates must be flat all over the floor; and the bearer joists must be made to give proper support to the floor plates everywhere by means of packings.

The end wall plates should return into each side wall, as shown in Fig. 38. They should also form an "outer" section, lying beneath the adjacent floor plates at the main seams. By this means they will be "tailed down" against the overturning effects of the cantileverage through the weight of the contained liquid on the adjacent plates, and without subjecting the rivets of the seam to lifting tension—as might happen if the end-plates formed an "inner" section.

If vertical seams are necessary in the end wall plates, corresponding with the subsidiary seams in the floor plates, they should be of the same type as the subsidiary seams—that is, preferably, of single-riveted, double-covered butt joints. The outer cover may, of course, be stopped at the curb angle, and the inner cover at the lap of the first main seam.

CHAPTER IV

WALLS OF RECTANGULAR TANKS

33. *Stayed Walls of Rectangular Tanks.*—We will now consider the various ways in which the side walls of rectangular tanks may be stayed against the outward pressure of the contained liquid in cases where they cannot be provided with adequate stability by any other suitable means.

Many unintelligent and ineffective methods of staying are in use; several others, reasonably sound in principle, are frequently so misapplied as to become worse than useless; and yet others, though obviously simple and practicable, are altogether ignored. It will, therefore, be more expeditious and satisfactory to first examine the facts as regards conditions and requirements, rather than to discuss the different methods in turn. With the latter course, we could only compare the various methods with each other, whereas by examining the facts we shall obtain a basis, or standard, to which we may then refer each method for comparison. The merits and demerits of the various methods will thus be in no way subject to personal opinion, but will stand out clearly as facts—any particular method being good (or the reverse) according as it meets (or does not meet) the conditions and fulfils the requirements, in a satisfactory manner for a reasonable cost.

The loading to be resisted is, primarily, the horizontal outward pressure of the contained liquid against the vertical wall. Sometimes (as will be seen presently) there are other additional factors in the loading, but these we may leave for treatment in due course. The intensity of the pressure in a liquid being directly proportional to the depth below the surface, it follows that the intensity of the pressure upon a side wall of a tank varies uniformly from zero at the surface of the liquid to a maximum at the base of the wall,

the resultant pressure upon a strip of the wall bounded by two cross-sections acting at a height (measured from the base of the wall) equal to one-third of the liquid head above the base. Under the action of varying heads, therefore, even were the density of the liquid constant, the loading on the side walls will vary both in magnitude and disposition. The magnitude and influences of the loading for a given head of a particular liquid may, of course, be readily estimated; and in actual structures it is necessary to provide for the most severe conditions likely to arise.

To resist this loading we have a vertical wall, which may possess either: (1) No dependable stability of its own to resist overturning; (2) some degree of partial stability; or (3) sufficient stability without assistance from stays or bracing.

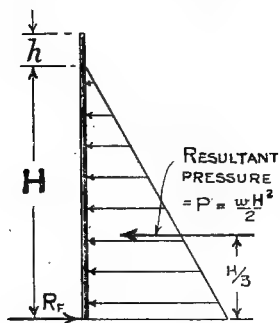


FIG. 42.

Of these three cases, the third was described and investigated* in pp. 86-92 (Chapter III), so we have the first and second cases left for consideration, these comprising, perhaps, the bulk of ordinary practice.

For the moment we shall assume that the side wall is in all cases connected with the tank floor in such manner as to prevent horizontal movement at the foot of the wall. Presently it will be shown that, in some circumstances, such connection is scarcely required for stability, though it is, of course, always necessary to prevent leakage.

In the first case, then, the conditions are as shown in Fig. 42, and it is obvious that, unless some additional resistance be provided, the wall will be overturned by the action of the unbalanced outward thrust. These conditions are practically realised in the case of an ordinary rectangular steel tank having plane walls connected to a plane floor by means of angle bars, as indicated at (a) in Fig. 34.

34. *Staying by means of Top Curb.*—The necessary additional resistance to overturning could be provided at the brim, by means of a curb possessing adequate strength against bending in the horizontal plane, as shown at (a) in Fig. 43, and this method might

with advantage be more widely employed than it is. A curb is almost invariably provided for some purpose—even if only to form a “finish”—and it may be turned to useful account. For a small increase in the cost of the actual curb, real advantage could often be secured, giving appreciable saving in the cost of the side walls. Lateral deflection of the curb, within reasonable limits, will not adversely affect the stability of the wall.

Each vertical strip of the wall sheeting may then be regarded as a beam, freely supported at the ends, and carrying a load which varies uniformly in intensity from zero at some point in the span to a maximum at one of the supports, as indicated at (b) in Fig. 43.

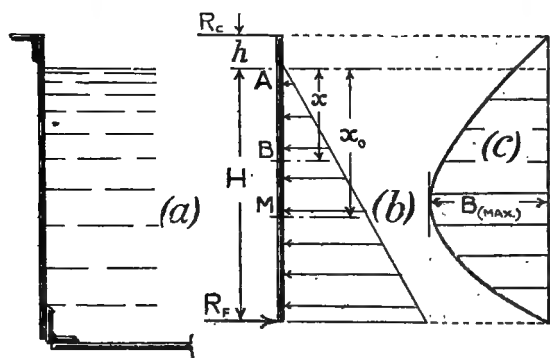


FIG. 43.

If H be the head or depth of the contained liquid in feet, the maximum intensity of the pressure will be $w H$ lb. per sq. ft., w being the weight of the liquid in pound per cubic foot.

Considering a vertical strip of wall 1 ft. in width, the average intensity of pressure will be $\frac{w H}{2}$ lb. per foot of height, and the total resultant pressure on the strip will be—

$$P = \left(\frac{w H}{2} \right) \times H = \left(\frac{w H^2}{2} \right) \text{ lb.}$$

The reaction at the curb will be—

$$R_c = P \left\{ \left(\frac{H}{3} \right) / (H + h) \right\} = \frac{w H^2}{2} \left\{ \frac{H}{3 (H + h)} \right\} = \frac{w H^3}{6 (H + h)},$$

and the reaction at the floor of the tank—

$$R_F = P \left\{ \left(\frac{2H}{3} + h \right) \right\} / (H + h) \left\} = \frac{wH^2}{2} \left\{ \frac{2H + 3h}{3(H + h)} \right\} = \frac{wH^2}{6} \left(\frac{2H + 3h}{H + h} \right).$$

The liquid pressure on a strip AB, x ft. in length, being $\frac{wx^2}{2}$ lb.,

the bending moment in the strip at any section B distant x ft. below the surface of the liquid will be—

$$\begin{aligned} B_B &= \left\{ R_C (h + x) - \frac{wx^2}{2} \left(\frac{x}{3} \right) \right\} = \left\{ \frac{wH^2}{6} \left(\frac{H}{H + h} \right) (h + x) - \frac{wx^3}{6} \right\} \\ &= \frac{w}{6} \left\{ \frac{H^3(h + x)}{(H + h)} - x^3 \right\}. \end{aligned}$$

For the conditions as indicated at (b) in Fig. 43, the bending moment diagram will be somewhat as shown at (c) in the same illustration.

To determine the greatest bending moment in the strip, the easiest course is first to locate the section at which maximum bending moment must occur—*i. e.* the section at which the transverse shear is zero. Let this section be at M, distant x_0 ft. below the surface of the liquid. Then, the liquid pressure on AM = $\frac{wx_0^2}{2}$, and for zero shear—

$$\frac{wx_0^2}{2} = R_C = \frac{wH^2}{6} \left(\frac{H}{H + h} \right),$$

whence—

$$x_0^2 = \frac{H^3}{3(H + h)}, \text{ and } x_0 = \sqrt{\frac{H^3}{3(H + h)}} = 0.577 \left\{ \sqrt{\frac{H^3}{H + h}} \right\}.$$

The maximum bending moment (at M) will thus be—

$$\begin{aligned} B_{\max} &= \left\{ R_C (h + x_0) - \frac{wx_0^2}{2} \left(\frac{x_0}{3} \right) \right\} = \left\{ \frac{wH^3}{6(H + h)} (h + x_0) - \frac{wx_0^3}{6} \right\} \\ &= \left\{ \frac{wH^3(h + x_0) - wx_0^3(H + h)}{6(H + h)} \right\} \end{aligned}$$

As a rule, it is well to provide for the possibility that the tank may be filled to the brim, in which case $h = 0$. Then the reactions will be—

$$R_C = \frac{wH^2}{6}; \text{ and } R_F = \frac{wH^2}{3}.$$

The depth of the section M will be—

$$x_0 = \sqrt{\left(\frac{H^2}{3}\right)} = 0.577 H;$$

the bending moment at any section B will be—

$$B_B = \frac{w}{6} (H^2 x - x^3) = \frac{w x}{6} (H^2 - x^2);$$

and the maximum bending moment (at M)—

$$B_{\max} = \frac{w H^3 x_0 - w x_0^3 H}{6 H} = \frac{w x_0}{6} (H^2 - x_0^2),$$

which, on inserting the value of $x_0 (= 0.577H)$ becomes—

$$B_{\max} = \frac{0.577 w H}{6} \left(\frac{2 H^2}{3}\right) = 0.064 w H^3.$$

If the contained liquid be water, or other liquid of equal density, w may be taken as 62 lb. per cub. ft., and the maximum bending moment will then be—

$$B_{\max} = 0.064 \times 62 H^3 = 3.977 H^3,$$

which, for all practical purposes, may be taken as—

$$B_{\max} = 4 H^3 \text{ ft.-lb.} = \frac{4 H^3 \times 12}{2240} = \frac{3 H^3}{140} \text{ in.-tons.}$$

With plate thickness t (in inches), the section modulus of a strip 1 ft. in width will be: $\frac{1}{6} \times 12 \times t^2 = 2t^2$, and taking a maximum permissible working stress of 7.5 tons per sq. in., the resistance moment of the strip will be: $2t^2 \times 7.5 = 15t^2$ in.-tons.

Equating the resistance moment with the maximum bending moment—

$$15 t^2 = \frac{3 H^3}{140}; \text{ whence } t^2 = \frac{H^3}{700},$$

and
$$t = \sqrt{\frac{H^3}{700}} = \frac{\sqrt{H^3}}{26.45} = 0.0378 \sqrt{H^3}.$$

Also, by transposition—

$$H^3 = 700 t^2; \text{ whence } H = \sqrt[3]{700 t^2} = 8.879 (\sqrt[3]{t^2}).$$

Inserting appropriate stock values for t , the following table,
H

comparable with that given on page 90 for the wall strip flanged in one piece with the floor, may be constructed—

$t =$ Thickness of Plate in Inches.	Calculations.	H = Maximum Permissible Head in Feet.
$\frac{1}{4}$	$H = \sqrt[3]{\frac{700}{16}} = \frac{8.8790}{2.5198}$	3.52
$\frac{5}{16}$	$H = \sqrt[3]{\frac{700 \times 25}{256}} = \frac{25.9625}{6.3496}$	4.09
$\frac{3}{8}$	$H = \sqrt[3]{\frac{700 \times 9}{64}} = \frac{18.4689}{4}$	4.62
$\frac{7}{16}$	$H = \sqrt[3]{\frac{700 \times 49}{256}} = \frac{32.4911}{6.3496}$	5.12
$\frac{1}{2}$	$H = \sqrt[3]{\frac{700}{4}} = \sqrt[3]{175} = 5.593$	5.59
$\frac{9}{16}$	$H = \sqrt[3]{\frac{700 \times 81}{256}} = \frac{38.4174}{6.3496}$	6.05
$\frac{5}{8}$	$H = \sqrt[3]{\frac{700 \times 25}{64}} = \frac{25.9625}{4}$	6.49
$\frac{11}{16}$	$H = \sqrt[3]{\frac{700 \times 121}{256}} = \frac{43.9166}{6.3496}$	6.92
$\frac{3}{4}$	$H = \sqrt[3]{\frac{700 \times 9}{16}} = \frac{18.4689}{2.5198}$	7.33

Comparing these depths with those tabulated on page 90, it will be seen that the permissible depth for each plate thickness is about 50 per cent. more with the arrangement here discussed than with the flanged wall arrangement of Figs. 38 and 39.

In an actual tank the wall is subjected to the pressure of the liquid from the top of the angle bar only, as indicated in Fig. 44, the angle bar being amply sufficient to take the liquid pressure acting upon its upstanding limb in addition to providing the reaction for the base of the wall. Moreover, being riveted to the angle bar, the wall plate will be rather more favourably placed for resisting the bending action than we have calculated upon, owing to the lower end having some slight (though not practically determinate) degree of partial fixity instead of being freely supported as we have assumed.

adding to the tabulated permissible depths the height of the up-standing limb of the angle bar which connects the wall to the floor, to obtain the total permissible height from the floor of the tank to the brim. Indeed, where the circumstances are positively known to be favourable, an even larger concession may be made, the tabulated depths being increased by such a distance more than the limb of the angle bar as may be properly and truly justifiable, having regard to the facts, to allow for the slight fixity of the plate at its lower end. These matters are best left to the discretion of competent and responsible designers, for application according to the facts and circumstances of particular cases. Any attempt at generalisation must inevitably be liable to prove misleading, since the calculations would be based upon assumptions which could not be equally realised in all cases. It will, of course, be clear that, even in the most favourable circumstances, the permissible addition to the tabulated depth will seldom be more than an inch or two beyond the height of the up-standing limb of the angle bar; the object in referring to it here is to show that, in cases where the tabulated depth corresponding to a convenient plate thickness gives a capacity so slightly less than that required that a few inches of additional depth would suffice to make up the deficiency, it may be possible to justify the use of the convenient plate thickness, and thus to avoid increasing the cost of the tank walls, either by adopting the next greater stock thickness for the plates throughout, or by providing additional staying or bracing.

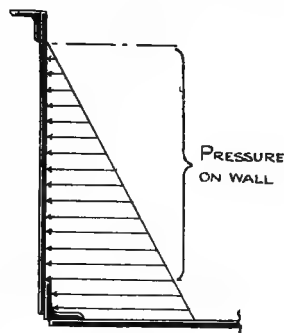


FIG. 44.

The necessary transverse support at the brim may be provided either by making the curb itself sufficiently strong to span from end to end of the tank, or by using a lighter curb in conjunction with some convenient and effective form of trussing, staying, or bracing. Simple and economical methods for dealing with this point are discussed and illustrated in Chapter V, the same methods being suitable, with no more than slight modification and adaptation, for use with several distinct forms of wall support.

One objection to the arrangement discussed above should be mentioned in passing. A more or less considerable direct tension is applied to the rivets which connect the wall plates to the floor angle bar, and it is always undesirable that tension should be applied to rivets, more particularly where the rivets are required to maintain tightness against leakage of a liquid or gas. It is not a serious objection, for in spite of the enormous numbers of tanks which are constructed in this way, one never hears of even isolated cases in which trouble has arisen from this cause. It will be clear, however, that in some circumstances such tension might be at least undesirable, and we shall next proceed to show other methods

of supporting tank walls against horizontal pressure, with which tension in the rivets is minimised to the point of practical elimination.

35.—*Staying by Horizontal Rails.*

The necessary additional resistance to overturning of the wall might be provided at any other level, instead of at the brim. An instance of this is shown at (a) in Fig. 45, the upper support being there placed at the level of the resultant pressure when the tank is full to the brim. It may be thought that with this arrangement

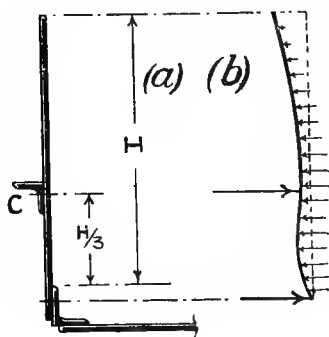


FIG. 45.

there would be no tension in the rivets connecting the wall plates with the floor angle, but such is not the case. By reason of its elasticity, the wall plating will deflect under loading; and the supporting rail C, no matter how stiff and strong it be, will deflect also. In consequence, the elastic line of the wall plates under pressure will be somewhat as indicated at (b) in Fig. 45, and some tension upon the base rivets is clearly inevitable with the rail C at the level shown, or in any higher position. By fixing the rail at a lower level, tension in the rivets could be prevented, and by lowering the rail still further, the wall plate could be made actually to press inwards against the floor curb angle—with the tank full to the brim. The conditions would be altered, however, as soon as the surface of the liquid fell below the brim of the tank.

With a rail giving support to the wall at some level between the

floor and brim, as indicated in Fig. 45, the sheeting is better stayed than in either of the arrangements previously discussed. Consequently, for the same plate thickness, a greater head is permissible with this method of staying; or, for a given depth of contained liquid, sheeting of less thickness may be used.

The case merits a complete and general investigation, so that full advantage may be taken of the benefits offered. Such an investigation will be given presently, together with the inferences which may be drawn for convenience in practical designing; but, for the sake of clearness and simplicity, it may be well to consider first a typical instance having specified particulars.

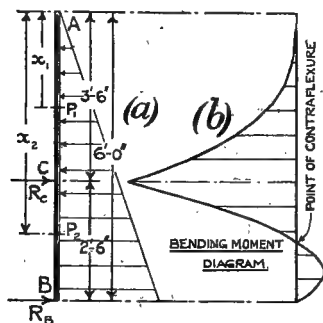


FIG. 46.

Let the depth be 6 ft., with the rail C at a height of 2 ft. 6 in. above the base of the wall, as indicated at (a) in Fig. 46, the contained liquid being water. We shall regard the wall as hinged—*i. e.* fixed in position, but not restrained as to direction—at B, and calculate for 1 ft. run of wall, assuming the wall plate to be in one piece, and of uniform thickness, from top to bottom.

The total liquid pressure on a 1 ft. strip will be—

$$\frac{wH^2}{2} = \frac{62.6 \times 36}{2} = 1119.6 \text{ lb., or (say) } 1120 \text{ lb.}$$

Taking moments about B, the outward force acting upon the rail C (per foot-run) will be—

$$R_C = \frac{1120 \times 2}{2.5} = 896 \text{ lb.,}$$

leaving $R_B = 1120 - 896 = 224 \text{ lb. per foot-run.}$

In the portion A C, the bending moment at any section P_1 , distant x_1 from A, may be found thus—

Total pressure acting

$$= \frac{w x_1^2}{2} \text{ ft.-lb. } (x_1 \text{ being in feet}).$$

Leverage of resultant $= \frac{x_1}{3}$ ft.

$$\therefore B_1 = \frac{w x_1^3}{6} = \frac{62.2 x_1^3}{6} = 10.368 x_1^3 \text{ ft.-lb.}$$

This, clearly, reaches a maximum at C, where its magnitude will be—

$$\begin{aligned} B_{1\max.} &= 10.368 \times (3.5)^3 = 10.368 \times 42.875 = 444.52 \text{ ft.-lb.} \\ &= \frac{444.52 \times 12}{2240} = 2.38 \text{ in.-tons.} \end{aligned}$$

In the portion B C the bending moment at any section P_2 , distant x_2 from A, will be—

$$\begin{aligned} B_2 &= \{10.368 x_2^3 - R_c (x_2 - A C)\} = \{10.368 x_2^3 - 896 (x_2 - 3.5)\} \\ &= (10.368 x_2^3 - 896 x_2 + 3136) \text{ ft.-lb.} \end{aligned}$$

Differentiating B_2 with respect to x_2 —

$$\frac{d B_2}{d x_2} = 31.1 x_2^2 - 896,$$

and for maximum bending moment, $\frac{d B_2}{d x_2} = 0$.

Hence, maximum bending moment occurs where $31.1 x_2^2 = 896$, and denoting this particular value of x_2 by x_0 , we have—

$$x_0^2 = \frac{896}{31.1} = 28.8, \text{ and } x_0 = \sqrt{28.8} = 5.37 \text{ ft.}$$

Inserting this value of x_2 in the expression for the bending moment between B and C—

$$\begin{aligned} B_{2\max.} &= \{10.368 \times (5.37)^3\} - \{896 (5.37 - 3.5)\} \\ &= (10.368 \times 154.854) - (896 \times 1.87) = 1605.53 - 1675.53 \\ &= -70 \text{ ft.-lb.} = -\frac{70 \times 12}{2240} = -0.375 \text{ in.-ton.} \end{aligned}$$

The magnitude of this bending moment is less than that at C,

and hence, working on the latter, with the resistance moment = 15 t^2 in.-tons, as before—

$$t^2 = \frac{2.38}{15} = 0.16, \text{ whence } t = \sqrt{0.16} = 0.4 \text{ in.}$$

With this arrangement, therefore, $\frac{7}{16}$ in. plates would be sufficient for a depth of 6 ft., as compared with 5.12 ft. for a curb at the brim (see page 98), and 3.44 ft. with the flanged plate arrangement of Fig. 38 (see page 90).

The diagram of bending moments on the plate is shown at (b) in Fig. 46, there being a point of contraflexure between the rail C and the base of the wall.

Now, it will be clear that if the rail C were lowered, the bending

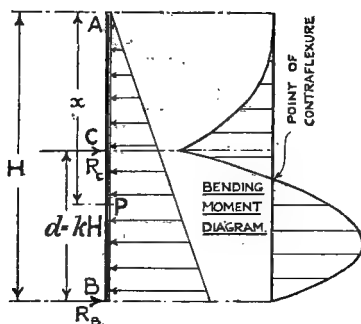


FIG. 47.

moment at C would be increased, and that between B and C diminished, giving a still greater disparity. On the other hand, if the rail C were raised, the maximum bending moments would become more and more nearly equal until, with the rail above a certain level, the maximum bending moment between B and C would preponderate over that at C. Obviously, the most economical arrangement will be that in which the bending moment at C is equal to the greatest bending moment occurring in the range B C, and to locate the position of the rail to give this, a general treatment is necessary, a convenient line of argument being as follows—

With the conditions as indicated in Fig. 47, for no bending moment at B, the pressure upon the rail C (per foot-run)—

$$= R_c = \frac{w H^3}{6 d}.$$

Let $d = K H$, then—

$$R_c = \frac{w H^3}{6 K H} = \frac{w H^2}{6 K}.$$

The bending moment at C will be—

$$\begin{aligned} B_c &= \frac{w (A C)^3}{6} = \frac{w (H - d)^3}{6} = \frac{w (H - K H)^3}{6} \\ &= \frac{w H^3}{6} (1 - K)^3 \text{ ft.-lb.} \end{aligned}$$

The bending moment at any section P between B and C will be—

$$\begin{aligned} B &= \left\{ \frac{w x^3}{6} - R_c (x - H + d) \right\} = \left\{ \frac{w x^3}{6} - \frac{w H^2}{6 K} (x - H + K H) \right\} \\ &= \left\{ \frac{w x^3}{6} - \frac{w H^2 x}{6 K} + \frac{w H^3}{6 K} - \frac{w H^3}{6} \right\} \text{ ft.-lb.} \end{aligned}$$

Differentiating B with respect to x —

$$\frac{d B}{d x} = \frac{w x^2}{2} - \frac{w H^2}{6 K},$$

and for maximum bending moment, $\frac{d B}{d x} = 0$, whence—

$$\frac{w x^2}{2} = \frac{w H^2}{6 K}, \text{ or } x_0^2 = \frac{H^2}{3 K}, \text{ and } x_0 = \frac{H}{\sqrt{3 K}}.$$

Inserting this value of x in the expression for B—

$$\begin{aligned} B_{\max.} &= \frac{w H^3}{18 \sqrt{3 K^3}} - \frac{w H^3}{6 \sqrt{3 K^3}} + \frac{w H^3}{6 K} - \frac{w H^3}{6} \\ &= \frac{w H^3}{6} \left(\frac{1}{3 \sqrt{3 K^3}} - \frac{1}{\sqrt{3 K^3}} + \frac{1}{K} - 1 \right), \end{aligned}$$

and equating this with the expression for the bending moment at C (bearing in mind that the two moments are of opposite sense)—

$$\frac{w H^3}{6} \left(\frac{1}{3 \sqrt{3 K^3}} - \frac{1}{\sqrt{3 K^3}} + \frac{1}{K} - 1 \right) = - \frac{w H^3}{6} (1 - K)^3,$$

which, on simplification, becomes—

$$1 - 3 + 3 \sqrt{3 K} = 3 \sqrt{3 K^3} \{1 - (1 - K)^3\},$$

whence $3 \sqrt{3 K^3} \{1 - (1 - K)^3\} - 3 \sqrt{3 K} + 2 = 0$,

or $\sqrt{K^3} \{1 - (1 - K)^3\} - \sqrt{K} + 0.385 = 0$.

This equation, in the form given, may be readily solved by means of a graph on squared paper, giving $K = 0.524$, whence—

$$d = 0.524 H.$$

With the rail fixed at this level, the maximum bending moment between B and C will be equal in magnitude to that at C; hence, the latter (being more easily calculated) may be used for the purposes of designing.

The rail should be fixed so that its axis (*i. e.* the centre of gravity of its cross-section) is at the level d , the dimensions being measured as indicated in Fig. 48. Obviously, the plate is more favourably circumstanced with this than with any other disposition of the rail C.

Returning to the case of a 6 ft. depth, with the more effective placing of the rail—

$$BC = d = 6 \times 0.524 = 3.14 \text{ ft.}, \text{ and } AC = 2.86 \text{ ft.}$$

$$\begin{aligned} \text{Hence } B_c &= \frac{w (2.86)^3}{6} = \frac{62.2 \times 23.3937}{6} = 241 \text{ ft.-lb.} \\ &= \left(\frac{241 \times 12}{2240} \right) \text{ in.-tons.} \end{aligned}$$

With the resistance moment = $15 t^2$ in.-tons, as before—

$$t^2 = \frac{241 \times 12}{2240 \times 15} = 0.0861, \text{ and } t = \sqrt{0.0861} = 0.29 \text{ in.},$$

which renders a $\frac{5}{16}$ in. plate ample, with a reasonable allowance for corrosion, beyond mere strength requirements.

The position of the section at which maximum bending moment occurs between B and C is easily found by inserting the value of K in the expression for x_0 resulting from $\frac{dB}{dx} = 0$. Thus—

$$\begin{aligned} x_0 &= \frac{H}{\sqrt{3K}} = \frac{H}{\sqrt{3 \times 0.524}} = \frac{H}{\sqrt{1.572}} = \frac{H}{1.256} \\ &= 0.798 H, \text{ or (very nearly) } 0.8 H. \end{aligned}$$

Since $d = 0.524 H$, by subtraction, $AC = 0.476 H$, and the bending moment for purposes of design will be—

$$B = \frac{w (0.476 H)^3}{6}.$$

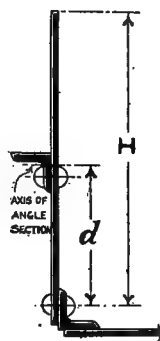


FIG. 48.

which, for water, becomes—

$$B = \frac{62.2 \times 0.10785 H^3}{6} = 1.1181 H^3 \text{ ft.-lb.}$$

$$= \frac{1.1181 \times 12}{2240} (H^3) \text{ in.-tons.}$$

With the resistance moment = $15 t^2$ —

$$t^2 = \frac{1.1181 \times 12 \times H^3}{2240 \times 15} = 0.000399 H^3,$$

whence $t = \sqrt{0.000399 H^3} = 0.02 (\sqrt{H^3}) \text{ in.,}$

and $H^3 = \frac{2240 \times 15 \times t^2}{1.1181 \times 12} = 2504 t^2,$

whence $H = \sqrt[3]{2504 t^2} = 13.58 (\sqrt[3]{t^2}).$

Inserting appropriate stock values for t , the following table, comparable with those given in pages 90 and 98, may be constructed—

$t =$ Thickness of Plate in Inches.	Calculations.	$H =$ Maximum Permissible Head in Feet.
$\frac{1}{4}$	$\sqrt[3]{\frac{2504}{16}} = \sqrt[3]{156.5}$	5.39
$\frac{5}{16}$	$\sqrt[3]{\frac{2504 \times 25}{256}} = \sqrt[3]{244.5}$	6.25
$\frac{3}{8}$	$\sqrt[3]{\frac{2504 \times 9}{64}} = \sqrt[3]{352.1}$	7.06
$\frac{7}{16}$	$\sqrt[3]{\frac{2504 \times 49}{256}} = \sqrt[3]{479.3}$	7.83
$\frac{1}{2}$	$\sqrt[3]{\frac{2504}{4}} = \sqrt[3]{626}$	8.56
$\frac{9}{16}$	$\sqrt[3]{\frac{2504 \times 81}{256}} = \sqrt[3]{792.3}$	9.25
$\frac{5}{8}$	$\sqrt[3]{\frac{2504 \times 25}{64}} = \sqrt[3]{978.1}$	9.93
$\frac{11}{16}$	$\sqrt[3]{\frac{2504 \times 121}{256}} = \sqrt[3]{1183.5}$	10.58
$\frac{3}{4}$	$\sqrt[3]{\frac{2504 \times 9}{16}} = \sqrt[3]{1408.5}$	11.20

On comparison, it will be seen that this arrangement permits

depths more than 50 per cent. in excess of those for the curb at the brim, and more than double those for the wall plates flanged in one piece with the floor plates.

36.—*Effects of Variations in Liquid Head.*—With regard to the arrangement of Fig. 48, the question arises as to whether the wall sheeting may be more severely stressed with the tank partly, instead of completely, filled. For a tank of given depth, and having the rail C fixed at the most economical level for the maximum head, it is obvious that the outward pressure of the liquid must diminish as the liquid head is reduced. On the other hand, however, since the rail C remains at its fixed level, the reaction R_c will be reduced as the surface of the liquid falls.

The net bending moment in the sheeting between B and C is the difference between the moments due to these two opposing influences, and since the inward and outward moments follow different laws, it is not clear, from a superficial view of the facts, that the maximum bending moment in the sheeting with the tank full will of necessity be more than that with the surface of the contained liquid standing at some lower level.

Moreover, the bending action changes in character as the liquid head varies. With the tank full, there is a point of contraflexure in the wall sheeting, and as the surface of the liquid falls between A and C, the point of contraflexure rises towards C. The surface of the liquid and the point of contraflexure arrive at C simultaneously, but the contrary bending moment due to the pressure of the liquid upon the portion of the sheeting above C, acting as a cantilever—which moment has become less and less as the liquid surface fell from A towards C,—vanishes as the surface of the liquid reaches C. As soon as the liquid surface reaches C, and for all less heads, the case becomes practically the same as that indicated in Fig. 43.

From this it follows that as the liquid surface varies between A and C (Fig. 47), the stress at the wetted skin of the wall sheeting at C varies between the maximum working stress in tension and zero, while the stress at the outer skin varies between maximum compression and zero. The same variation in the level of the liquid surface causes the stress at that section of the wall sheeting which marks the point of contraflexure with the tank brim-full to vary between zero and a considerable compression at the wetted

less than H , the bending moment at C will be less than the corresponding bending moment with the tank full to the brim.

With the surface level between A and C , as in Fig. 49, the bending moment at any section P between B and C will be—

$$B = \left\{ \frac{w x^3}{6} - R_c (x - h + K H) \right\} = \left\{ \frac{w x^3}{6} - \frac{w H^2 q^3 x}{6 K} + \frac{w H^3 q^4}{6 K} - \frac{w H^3 q^3}{6} \right\}$$

$$= \frac{w}{6} \left\{ x^3 - \frac{H^2 q^3 x}{K} + \frac{H^3 q^4}{K} - H^3 q^3 \right\}.$$

Differentiating B with respect to x —

$$\frac{dB}{dx} = \frac{w}{6} \left\{ 3x^2 - \frac{H^2 q^3}{K} \right\},$$

and for maximum bending moment, $\frac{dB}{dx} = 0$, whence, maximum bending moment occurs at the section where—

$$3x^2 = \frac{H^2 q^3}{K}.$$

or, denoting this particular value of x as x_0 —

$$x_0 = \sqrt{\frac{H^2 q^3}{3K}} = 0.7976 H \sqrt{q^3}.$$

In passing, it is worth noting that this section of maximum bending moment remains at practically the same level in any given tank throughout almost the whole range of possible variation in the level of the liquid surface.

The distances x and x_0 are, of course, measured downwards from the surface of the liquid, as indicated in Fig. 49, and, as this surface is to vary, it will be well to locate the sections at which the maximum bending moments occur with reference to some fixed point—say, A , the brim of the tank.

Then, the distances of the sections below A will be given by—

$$x_1 = x_0 + (H - h) = x_0 + H(1 - q),$$

and inserting the value of x_0 —

$$x_1 = H(0.7976 \sqrt{q^3} + 1 - q),$$

which may be written as : $x_1 = H(a)$.

Giving to q the values 1, 0.9, 0.8, ... 0.2, 0.1, and 0, the

corresponding values of a may be calculated. These are as shown in the accompanying table, and indicate that the point of maximum bending moment *rises* by 0.03 H. as the surface of the liquid falls

q .	a .	q .	a .
1.0	0.7976	0.4	0.8018
0.9	0.7812	0.3	0.8311
0.8	0.7707	0.2	0.8714
0.7	0.7672	0.1	0.9252
0.6	0.7707	0	1.0
0.5	0.7820		

from H to 0.7 H, after which, further falls of the liquid surface are accompanied by very slight *lowerings* of the section at which maximum bending moment occurs, until the liquid head has been reduced to 0.2 H, when the point of maximum bending moment falls almost as rapidly as does the liquid surface.

In a tank 10 ft. in depth, therefore, lowering the liquid surface from the brim to 3 ft. below, the section of maximum bending moment would be *raised* 3.65 in. Further reductions of head, down to a depth of only 3 ft., would *lower* the section of maximum bending moment by about 7.66 in., bringing it only about 4 in. below the level at which it stood with the tank full to the brim. Obviously, the effects with heads less than 0.3 H are unimportant.

Substituting $0.7976 H \sqrt{q^3}$ for x in the expression for the bending moment, a general relation may be obtained, giving the maximum bending moment between B and C for any level of the liquid surface. Thus—

$$B_{\max} = \frac{w H^3}{6} \left\{ (0.7976 \sqrt{q^3})^3 - \frac{0.7976 q^{4.5}}{0.524} + \frac{q^4}{0.524} - q^3 \right\}.$$

This expression could be differentiated with respect to q , and the value of q giving the greatest maximum bending moment thus determined. The expressions obtained would, however, be rather awkward to deal with, and this may be left as an exercise for those having sufficient enthusiasm and leisure.

Another method, probably more convincing, will suit our purpose equally well.

Take $H = 10$ ft., and plot the curve $\left(y = \frac{w \cdot x^3}{6} \right)$ of moments due

to the liquid pressure, as indicated in Fig. 50. The single curve may be used for all values of h by taking the base-line as much *above* the horizontal through O as the liquid surface is to be *below* the brim. Thus, the moment curve for $h = 9$ ft. is the portion of the curve above the horizontal through 1; and so on.

The diagram of moment due to R_c may be drawn by means of a straight line joining the extremity of the liquid pressure curve with a point on the vertical axis $0.524 H$ (which is equal to 5.24 ft. in the case chosen) above the horizontal representing the floor of the tank. This, also, may be drawn for all heads on the single diagram, as shown.

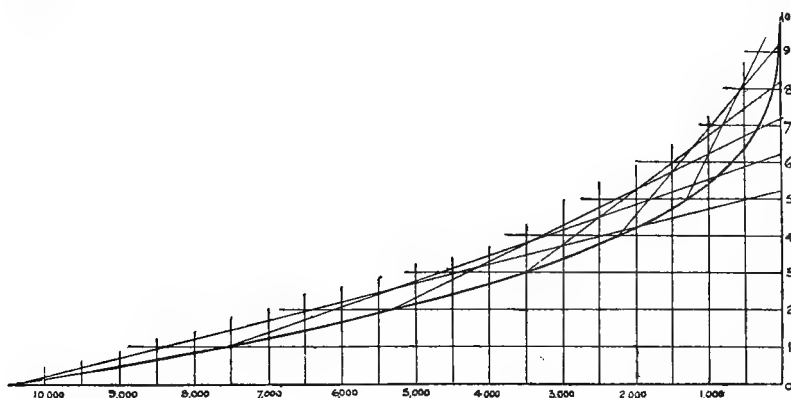


FIG. 50.

Bending moments are shown by the horizontal intercepts between the straight and curved moment diagrams, as indicated, for clearness, in Fig. 51, which relates to the case for $h = 0.9 H$.

A tangent to the curved line, parallel with the appropriate straight diagram line, will show the maximum bending moment between B and C for any particular value of h .

Every one interested in the matter should draw this diagram for himself. It is impossible to extract all the information by studying a complete diagram drawn by another.

Upon investigation it will be found that the maximum net bending moment between B and C is greatest when h is about $0.9 H$, its magnitude then being approximately 0.7 per cent. more than with the tank full to the brim—the liquid being supposed to have a specific gravity equal to that of water. This is so small an excess

that the relations previously obtained may be allowed to stand for all ordinary cases, and the permissible depths corresponding to stock plate thicknesses, as tabulated on page 106, may be used.

The sheeting does, of course, receive a certain degree of assistance from the restraint imposed at B by the ordinary riveted connection to the curb angle, and also from the resistance of the rail C to torsion, though such assistance is too uncertain and variable to be taken into account in the calculations. Such assistance may easily outweigh the effects of the increased loading, but with a tank of very great depth—or with a very heavy liquid to be contained—

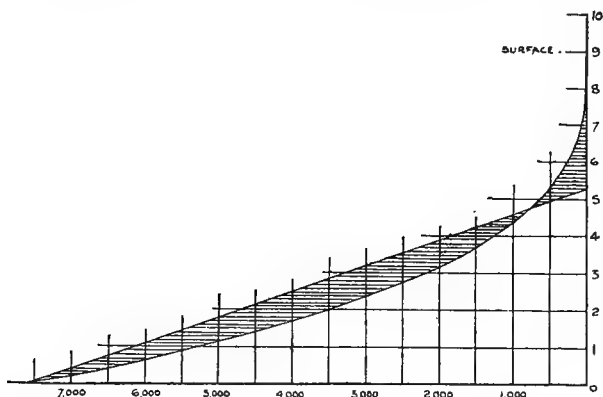


FIG. 51.

it might be well to provide some slight relief for the sheeting, rather than to increase its thickness throughout.

Generally, it will be found that sufficient relief may be obtained by lowering the rail C slightly—so that its axis lies at a height of about $0.5 H$ instead of $0.524 H$ —and providing it with a fairly wide vertical limb.

Lowering the rail C will assist the sheeting between B and C, and throw a slightly more severe bending moment upon it at C—as may be seen from a consideration of Fig. 50. In the nature of things, however, a tank can seldom be brimful, and hence the more severe loading at C is not likely to materialise. Moreover, the width of the rail C (vertically) will have the effect of distributing the reaction R_0 to some extent, and therefore the diagram of net bend-

ing moment will not have a sharp cusp at the level of C (as we have supposed), but will be rounded off, reducing the maximum intensity of the stresses in the sheeting at that level.

Though it should be unnecessary to do so, it may be well to emphasise the fact that the permissible depths tabulated for stock plate thicknesses in the preceding examples are for water (or other liquids of equal density), weighing 62·2 lb. per cub. ft., and at ordinary atmospheric pressures. For heavier liquids, and for cases in which additional pressure may be applied—either by the liquid standing in a rising pipe above the top of the tank, with the roof tight against leakage, or by any other means—the tabulated values do not apply. Though it is hoped they may be of use in a considerable range of practical cases, these tabulated permissible depths are given primarily for the purpose of showing the comparative merits of the various methods of staying the wall sheeting of tanks.

37.—*Walls with 'Several Horizontal Rails.*—In view of the advantageous results which, as we have shown, are to be obtained by placing the horizontal rail C at a suitable level, it would seem reasonable to suppose that further advantage might be gained from the use of two such rails at appropriate levels—or even, with very deep tanks, of several rails.

To a certain extent this is true, but, as will be seen presently, the arrangement of the sheeting must be altered if any appreciable saving is to be effected.

With two horizontal rails, one might be placed at the brim to form a curb, and the other at some intermediate level between the brim and floor, as indicated in Fig. 52; or both rails might be between the brim and floor, as in Fig. 53, leaving a portion of the sheeting to act as a cantilever above the upper rail.

We will consider the former of these arrangements, afterwards disposing of the latter by means of inferences to be drawn from the results of our investigation. As in the preceding cases, we shall ignore any partial fixity which there may be at the base connection, the wall being regarded as hinged—*i. e.* fixed in position, but not restrained as to direction—at B.

The magnitudes of the reactions at A, B, and C will depend, of course, upon the relative movements permitted by the supports

at those levels, as well as upon the position of the rail C. We will first investigate the question on the assumption that no appreciable movement is possible at either A, B, or C; and afterwards we may consider the effects of such movements as might be expected in actual cases.

First it is necessary to determine the deflections of the wall plate, acting as a beam freely supported at A and B, under the action of the liquid pressure only, and supposing the rail C removed. It is, perhaps, easiest for this purpose to imagine the beam-strip as separated into two cantilevers, the cut being made

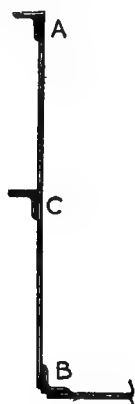


FIG. 52.

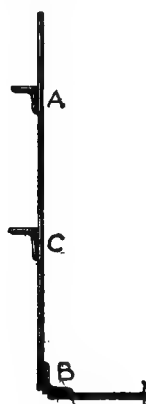


FIG. 53.

at the section where maximum deflection occurs, each cantilever portion being treated as "fixed" at that section. The conditions of loading and support for the complete beam-strip are indicated at (a) in Fig. 54, and those for the two portions to be treated as cantilevers at (b) and (c) in the same illustration.

Then, from (b) in Fig. 54, the symbols retaining the meanings assigned to them for the cases already treated, the bending moment at any section P, distant x from O, will be—

$$B_P = \left\{ \frac{w}{6} H^2 (H_1 - x) - \frac{w (H_1 - x)^3}{6} \right\}$$

$$= \frac{w}{6} \left\{ H_1 (H^2 - H_1^2) - x (H^2 - 3H_1^2) - 3H_1 x^2 + x^3 \right\},$$

whence

$$\frac{d}{dx} y = \frac{1}{EI} \int B_p dx =$$

$$\begin{aligned} \frac{w}{6EI} \left\{ x H_1 (H^2 - H_1^2) - \frac{x^2}{2} (H^2 - 3H_1^2) - H_1 x^3 + \frac{x^4}{4} + (C=0) \right\} \\ = \frac{w}{24EI} \left\{ 4x H_1 (H^2 - H_1^2) - 2x^2 (H^2 - 3H_1^2) - 4x^3 H_1 + x^4 \right\}; \end{aligned}$$

and integrating again—

$$\begin{aligned} y = \frac{w}{24EI} \\ \left\{ 2x^2 H_1 (H^2 - H_1^2) - \frac{2x^3}{3} (H^2 - 3H_1^2) - x^4 H_1 + \frac{x^5}{5} + (C=0) \right\} \\ = \frac{w}{360EI} \{ 30x^2 H_1 (H^2 - H_1^2) - 10x^3 (H^2 - 3H_1^2) - 15x^4 H_1 + 3x^5 \} \end{aligned}$$

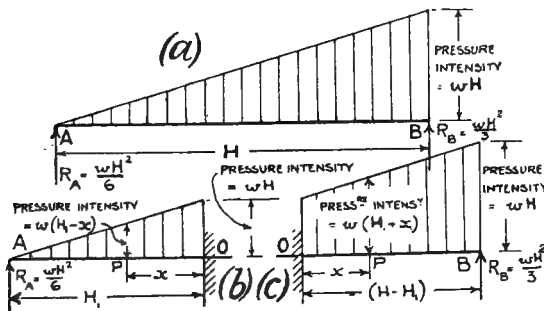


FIG. 54.

When $x = H_1$, y will be a maximum, and its magnitude will then be—

$$y_{(\max)} = \delta =$$

$$\begin{aligned} \frac{w}{360EI} \{ 30 H_1^3 (H^2 - H_1^2) - 10 H_1^3 (H^2 - 3H_1^2) - 15 H_1^5 + 3 H_1^5 \} \\ = \frac{w}{360EI} \{ 30 H_1^3 H^2 - 30 H_1^5 - 10 H_1^3 H^2 + 30 H_1^5 - 15 H_1^5 + 3 H_1^5 \} \\ = \frac{w}{360EI} \{ 20 H_1^3 H^2 - 12 H_1^5 \}. \end{aligned}$$

Another value of δ may be obtained by reasoning from (c) i Fig. 54, and on equating these two values, the ratio borne by H to H will be obtained, and the section at which maximum deflection occurs will thus be located. To obtain an expression for the bending moment at any section P , distant x from O in (c), Fig. 54, however it is necessary to know the leverage of the resultant liquid pressure to the right of P , and, in order that our treatment may be reasonable complete, it will be well to establish this leverage in passing.

The problem is merely to determine the horizontal distance between the centre of gravity of the area shown in Fig. 55 and the axis ZZ . Dividing the area, by means of the dotted line, into

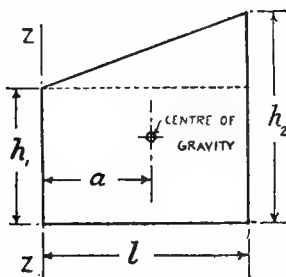


FIG. 55.

rectangle and a triangle, the moment of the area about the axis ZZ will be—

$$\begin{aligned} M &= \left\{ l h_1 \left(\frac{l}{2} \right) + \frac{l (h_2 - h_1)}{2} \left(\frac{2l}{3} \right) \right\} = \frac{l^2 h_1}{2} + \frac{l^2 (h_2 - h_1)}{3} \\ &= \frac{3 l^2 h_1 + 2 l^2 h_2 - 2 l^2 h_1}{6} = \frac{l^2}{6} (2 h_2 + h_1). \end{aligned}$$

The area is : $A = l \left(\frac{h_1 + h_2}{2} \right)$, and hence the required leverage will be—

$$a = \frac{M}{A} = \frac{l^2}{6} (2 h_2 + h_1) \times \frac{2}{l (h_2 + h_1)} = \frac{l (2 h_2 + h_1)}{3 (h_2 + h_1)}.$$

Substituting for h_2 and h_1 the intensities of pressure at B and respectively, and for l the distance $(H - H_1 - x)$, the leverage of the resultant liquid pressure to the right of P will be—

$$\begin{aligned} a &= \frac{(H - H_1 - x) \{2wH + w(H_1 + x)\}}{3 \{wH + w(H_1 + x)\}} = \frac{(H - H_1 - x) (2H + H_1 + x)}{3 (H + H_1 + x)} \\ &= \left\{ \frac{2H^2 - HH_1 - H_1^2 - xH - 2xH_1 - x^2}{3 (H + H_1 + x)} \right\}. \end{aligned}$$

The average liquid pressure between P and B will be—

$$\frac{w}{2} \{H + H_1 + x\}$$

and this average pressure acting over the distance between P and B gives the total liquid pressure to the right of P as—

$$\frac{w}{2} (H + H_1 + x) (H - H_1 - x).$$

Hence, the bending moment at P will be—

$$B_p = \frac{w H^2}{3} (H - H_1 - x) - \frac{w}{2} (H + H_1 + x) (H - H_1 - x) \\ \left\{ \frac{2 H^2 - H H_1 - H_1^2 - x H - 2 x H_1 - x^2}{3 (H + H_1 + x)} \right\},$$

which, on simplification, gives—

$$\frac{d^2 y}{d x^2} = \frac{B_p}{E I} = \frac{w}{6 E I} (H^2 H_1 + x H^2 - H_1^3 - 3 x H_1^2 - 3 x^2 H_1 - x^3).$$

Integrating—

$$\frac{d y}{d x} = \frac{w}{6 E I} \left\{ H^2 H_1 x + \frac{x^2}{2} (H^2) - x H_1^3 - \frac{x^2}{2} (3 H_1^2) - x^3 H_1 - \frac{x^4}{4} + (C=O) \right\} \\ = \frac{w}{6 E I} \left\{ x H_1 (H^2 - H_1^2) + \frac{x^2}{2} (H^2 - 3 H_1^2) - x^3 H_1 - \frac{x^4}{4} \right\}$$

Again integrating—

$$y = \frac{w}{6 E I} \left\{ \frac{x^2 H_1}{2} (H^2 - H_1^2) + \frac{x^3}{6} (H^2 - 3 H_1^2) - \frac{x^4}{4} (H_1) - \frac{x^5}{20} + (C=O) \right\} \\ = \frac{w}{360 E I} \{ 30 x^2 H_1 (H^2 - H_1^2) + 10 x^3 (H^2 - 3 H_1^2) - 15 x^4 H_1 - 3 x^5 \}.$$

When $x = (H - H_1)$, y will be a maximum, and its magnitude will then be—

$$y_{(\max)} = \delta =$$

$$\frac{w}{360 E I} \{ 7 H^5 - 30 H^3 H_1^2 + 20 H^2 H_1^3 + 15 H H_1^4 - 12 H_1^5 \}$$

Equating the two values of δ —

$$20 H^2 H_1^3 - 12 H_1^5 = 7 H^5 - 30 H^3 H_1^2 + 20 H^2 H_1^3 + 15 H H_1^4 - 12 H_1^5. \\ \therefore 7 H^5 - 30 H^3 H_1^2 + 15 H H_1^4 = 0.$$

Writing $r H$ instead of H_1 (*i. e.* letting $H_1 = r H$)—

$$7 H^5 - 30 H^3 r^2 + 15 H^5 r^4 = 0.$$

$$\therefore H^5 (7 - 30 r^2 + 15 r^4) = 0.$$

The equation : $7 - 30 r^2 + 15 r^4 = 0$ may be solved, by plotting upon squared paper or otherwise, to give : $r = 0.52$; so that $H_1 = 0.52 H$.

Inserting this value of H_1 in the first expression for δ above—

$$\delta = \frac{4 w H^5}{360 E I} \left\{ 5(0.52)^3 - 3(0.52)^5 \right\} = \left\{ \frac{0.0065442 w H^5}{E I} \right\}.$$

To neutralise this deflection by means of a single concentrated load midway between A and B—

$$\frac{R_c H^3}{48 E I} = \frac{0.0065442 w H^5}{E I};$$

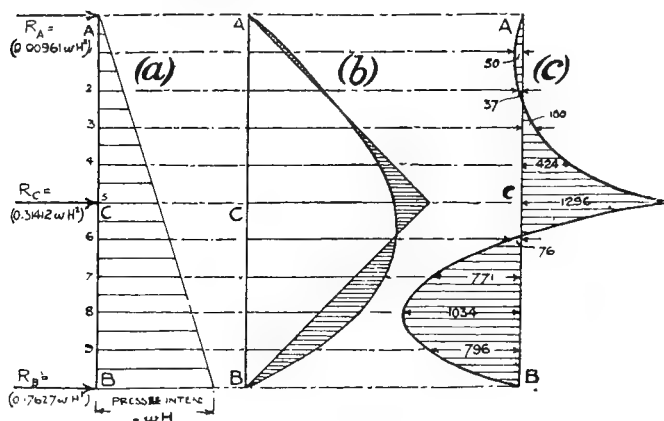


FIG. 56.

whence $R_c = 0.31412 w H^2$. This will reduce the reactions at A and B, each by $0.15706 w H^2$, giving—

$$\text{Net reaction at A} = \frac{w H^2}{6} - 0.15706 w H^2 = 0.00961 w H^2; \text{ and}$$

$$\text{Net reaction at B} = \frac{w H^2}{3} - 0.15706 w H^2 = 0.17627 w H^2.$$

The complete loading conditions for the continuous beam-strip are, then, as indicated at (a) in Fig. 56; the component and net bending moment diagrams as at (b); and the net bending moment diagram, reduced to a vertical axis, and the ordinates to an enlarged scale, for clearness and convenience of examination, as shown at (c) in the same illustration.

The figured ordinates of the diagram at (c) in Fig. 56 represent the bending moments, in foot-pounds, for the case in which $H = 10$ ft., calculated from the expressions—

$$B_{(A \text{ to } C)} = \left\{ \frac{w x^3}{6} - R_A x \right\} = \left\{ \frac{w x^3}{6} - 0.00961 w H^2 x \right\};$$

and

$$\begin{aligned} B_{(C \text{ to } B)} &= \left\{ \frac{w x^3}{6} - R_A x - R_C \left(x - \frac{H}{2} \right) \right\} \\ &= \left\{ \frac{w x^3}{6} - 0.00961 w H^2 x - 0.31412 w H^2 \left(x - \frac{H}{2} \right) \right\}. \end{aligned}$$

Taking the weight of the contained liquid as $w = 62.2$ lb. per cubic foot, these expressions may be simplified to—

$B_{(A \text{ to } C)} = \{x(10.37 x^2 - 60)\}$; and, $B_{(C \text{ to } B)} = \{x(10.37 x^2 - 2014) + 9770\}$, from which the figured ordinates may be calculated.

Now, if the bending moment diagram shown in Fig. 56 be compared with that (see Fig. 47) for the single rail at C, it will be found that there is practically no difference between them—certainly not sufficient to permit a reduction of $\frac{1}{16}$ in. in the plate thickness for ordinary tank-depths. Moreover, in the arrangement of Fig. 52 there are two rails as against one in the case of Fig. 47, and this extra cost would not be recovered.

The pressure upon the rail C—about 17.5 cwt. per foot-run with a tank 10 ft. in depth—is so much that, even with a very stiff (and therefore costly) rail, some outward movement of the plating at C would be inevitable. This would have the effect of reducing the bending moment at C, and increasing it between C and B, so that the plating, with this arrangement, might be subjected to more severe loading than that with the arrangement discussed in pp. 100–113.

Further, in view of the circumstances of loading and horizontal support, it would be practically impossible to estimate the extent of movement likely to occur at C with sufficient probability of truth to render any design for the sheeting, based upon such estimated movement, reasonably dependable.

No useful purpose would be served by raising the rail C—indeed, quite the reverse; nor would matters be improved by lowering the rail C, for the pressure upon it would be considerably increased,

and the degree of movement permitted in the sheeting more problematical than before.

With very deep tanks there might be a certain amount of advantage to be gained from the arrangement of Fig. 53, though the advantage would be lost as soon as the liquid surface fell to the level of the upper rail.

It would seem, therefore, that the only advantageous way to utilise two or more horizontal rail supports for the side-walls would be to let the sheeting lie in horizontal strakes, bearing upon a rail to the top and bottom, and not attempting to secure either continuity or uniformity of support.

38.—*Walls with Vertical Stiffeners.*—Vertical stays may be employed, instead of horizontal rails, to support the side wall against overturning.

This method (which is widely used) we shall now consider, though the author is of opinion that, as a rule, for rectangular tank of ordinary dimensions and proportions, the properly placed horizontal rail, as indicated in Fig. 48 (p. 105), provides the most satisfactory and economical form of wall support.

An arrangement in common use is shown in Fig. 57, the vertical stays being generally of an angle bar, riveted to the bottom and to the top curbs. Clearly, if the vertical stays are to act as beams, the top curb must be capable of providing the upper reactions, and should either possess sufficient strength and stiffness of their own for this purpose, or be themselves adequately stayed in some convenient manner.

At the outset we are met with the difficulty of determining as to how the wall plating acts in transmitting the horizontal pressure to the stays. Obviously, from Fig. 57, if it were not supported by the floor and top curbs, the plating would act simply as a beam between the stays, and the design would then be a simple matter. At the floor, however, the wall plating must be securely fastened to the curb angle, in order to prevent leakage, and the question is thus raised as to the effect, as regards nature and extent, which this has upon the action of the plating. No profound argument is needed to show that the ultimate effect of closely riveting the plating to the bottom curb must be to increase its strength and stiffness in its lower portions, and the question remaining is as to the

manner in which the additional strength is imparted to the plating, and the extent to which it may properly be relied upon in designing tanks for commercial purposes. Obviously, to design the plating on the basis of a beam strip spanning between the vertical stays, and subjected to the full intensity of pressure exerted by the contained liquid at the tank floor, would be to waste material, but it is not easy to see what allowance should be made for the additional strength imparted to the plating.

So far as the author is aware, no satisfactory solution of this

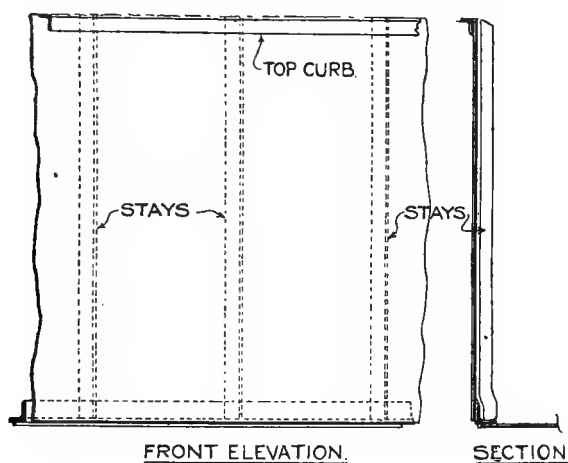


FIG. 57.

question, based upon rigid analysis, has yet been obtained; and it would seem unlikely that any such solution, applicable to the practical design of tank walls, will be found.

Various formulæ have been deduced for the stresses set up in flat plates supported or fixed at all their edges, but it is at least doubtful whether such relations can be employed, with any degree of reliability, in designing the wall plating for rectangular tanks. Most of these formulæ are based upon assumptions regarding the distribution of loading to give equal deflections (or, more strictly, the same deflection) in two beam strips of the plate intersecting at right angles; but while there can be no doubt that some such distribution of loading is automatically effected, the deflection

produced at any particular part of the plate must depend upon several factors which, from their nature, cannot be accurately foreseen, nor satisfactorily provided for, on a basis of assumption, in circumstances such as those occurring in tank work.

For instance, the actual and relative degrees of stiffness in the supports cannot fail to influence the local deflections in the plate, and since these degrees of stiffness are inevitably different at the different edges of the plate-panels—besides being variable from point to point along some edges while others are sensibly rigid throughout—it is practically impossible to deduce an expression which shall take proper account of the variations probable or possible in the framing of an ordinary tank. It is clear from Fig. 57 that the plate will be, to all intents and purposes, rigidly supported along the bottom curb, but the vertical stays and top curb will be subject to elastic deflection under loading. Moreover, even if the upper ends of the stays be fixed in position, the elastic lines of the stays will be unlike that of the top curb, for the loading upon the latter will be uniform—or, at least, symmetrical—while that acting upon the stays will increase from a minimum near their upper ends to a maximum near their lower ends.

Further, the plating of a tank wall of the type under discussion is always continuous over some of its supports (*viz.*, those vertical stays at which no seam in the plating occurs); subject to more or less partial restraint as to direction at others (the bottom curb); and perhaps almost freely supported at others (the top curb).

Thus it will be seen that irregularities and inequalities are possible in combinations covering a range so wide as to render it unlikely that any single expression could take them into account, even approximately, and remain usable in ordinary practical designing.

As an alternative, the author would suggest the following argument as being at least equally reasonable, while yielding a more simple basis for practical design.

Consider the panel of plating indicated in Fig. 58, freely supported along the edges A D, B C, and C D, and unsupported along A B, subjected to transverse loading such as would be applied by liquid pressure—*i. e.* varying uniformly in intensity from zero at A B to a maximum at C D. The supports at A D and B C correspond to the vertical stays of a tank wall, and that at C D to the bottom

curb. We are ignoring the lateral supporting effect of the top curb (along A B) because it is usually formed of a light angle bar only, which could not be relied upon to influence the distribution of the loading, especially as the height (A D) of the panel is generally about double of its width (A B), and the intensities of pressure in the neighbourhood of A B are small as compared with those nearer C D.

Now, even though the supports A D and B C may yield through elastic deflection, the movements in the portions near their lower ends, from this cause, will be exceedingly minute, while the plate may be taken as fixed in position along the edge C D.

Let us consider the effects of an element of pressure in some position such as E in Fig. 58. It will tend to cause deflections in many beam strips, two of which are indicated as F G and H K; but the resistance of F G to such deflection will be far more than that of H K, and hence a much greater proportion of the load at E will be taken by the strip along F G—indeed, strips parallel with F G up to about 1 ft. in length will be sensibly rigid. Similar conditions will, of course, obtain in the neighbourhood of the corner D, the plate being very stiff along such strips as $F_1 G_1$.

Proceeding inwards from C and D it is obvious that a stage will somewhere be reached at which the stiffness of the plate along a strip parallel to F G will be not more than that parallel with H K, and ultimately, at some horizontal strip such as S S, the plate will receive little or no assistance from the support along C D.

From observations made, the author is inclined to the view that this disappearance of the additional stiffness occurs, in ordinary cases, at a height (above the bottom curb) of about one-half to

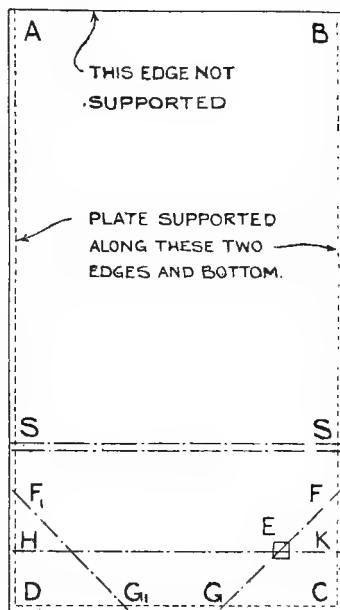


FIG. 58.

two-thirds of the distance between the vertical stays—that is to say, the panel may be designed on the basis of a narrow beam strip S S, subjected to the pressure intensity corresponding to a liquid depth B S, taking $SC = \frac{1}{2} CD$, and ignoring the greater intensities of pressure below S S. The author hopes, however, to make further observations and tests, and may have more to say on this point later.

We have considered the panel as freely supported along A D, B C, and C D, with the object of simplifying the discussion. The argument would, however, apply equally to the case of a plate continuous over the supports A D and B C, while the stiffness of the plate along strips as F G and $F_1 G_1$ will be so great that, in all probability, any partial fixity as to direction which the plate may receive through being riveted closely to the bottom curb (along C D) will not appreciably affect the inference for practical purposes.

Hence, in the design for an actual case, if the plating were not restrained as to direction at the supports A D and B C, the strip S S should be regarded as freely supported at its ends; while the same strip might be treated as “fixed” or “built-in” at its ends if the plating were continuous over the vertical stays, with all panels of equal width.

For the purpose of comparing the arrangement indicated in Fig. 57 with those dealt with in the preceding pages, let us consider the relationship existing between the liquid head and plate thickness for a given width of panel.

Taking the liquid head above S S (Fig. 58) in inches as H, and considering a beam-strip of the plating 1 in. in width at the level S S, the total pressure acting upon the strip will be $(w H L)$ lb., w being the weight of the liquid in pounds per cubic inch, and L the length of span S S in inches. The maximum bending moment in the strip—if it be continuous over the vertical stays—will be—

$$B = \left(\frac{w H L^2}{12} \right) \text{ in.-lb.} = \left(\frac{w H L^2}{12 \times 2240} \right) \text{ in.-tons.}$$

The strength modulus of the section will be $\left(\frac{t^3}{6} \right)$, where t is the thickness of the plate in inches; and taking a maximum permissible

stress $f = 7.5$ tons per sq. in., the resistance moment of the strip will be—

$$R = \left(\frac{7.5 t^2}{6} \right) = (1.25 t^2) \text{ in.-tons.}$$

Equating the resistance and bending moments—

$$\frac{w H L^2}{12 \times 2240} = 1.25 t^2,$$

whence—

$$H = \frac{12 \times 2240 \times 1.25 t^2}{w L^2} = \frac{33,600 t^2}{w L^2}.$$

If the contained liquid be water, taking w as $\left(\frac{62.2}{1728} \right)$ lb. per cub. in.—

$$H = \frac{33,600 t^2 \times 1728}{62.2 L^2} = 933,454 \left(\frac{t}{L} \right)^2.$$

As was to be expected, the permissible liquid head above SS varies directly with the square of the plate thickness, and inversely as the square of the panel width. Hence, plate thicknesses can only be tabulated with the corresponding liquid heads for a constant panel width. Such tables may easily be constructed, to cover the range of panel widths likely to be used, but there seems no need to include them here. A few typical instances will serve our purpose for comparison.

Taking the case of a $\frac{3}{8}$ in. plate with a span of 36 in.,

$$\left(\frac{t}{L} \right)^2 = \left(\frac{3}{8 \times 36} \right)^2 = \frac{1}{9216}.$$

whence—

$$H = \frac{933,454}{9216} = 101 \text{ in.}$$

Adding half the panel width (*i. e.* $\frac{36}{2} = 18$ in.), the total permissible height above the tank floor would be $101 + 18 = 119$ —or, say, 10 ft. A plate thickness of $\frac{3}{8}$ in. with the arrangement of Fig. 48 gave a permissible head of 7.06 ft., and hence, an additional 3 ft. of depth is obtained in return for the cost of the vertical stays at 3 ft. centres.

With $\frac{1}{4}$ in. plate over a 48 in. panel,

$$\left(\frac{t}{L}\right)^2 = \left(\frac{1}{4 \times 36}\right)^2 = \frac{1}{20,736},$$

whence—

$$H = \frac{933,454}{20,736} = 45 \text{ in.}$$

Adding half the panel with (*i. e.* $\frac{48}{2} = 24$ in.), the total permissible head above the tank floor would be $45 + 24 = 69$ in. = 5.75 ft. A plate thickness of $\frac{1}{4}$ in. with the arrangement of Fig. 48 gave a permissible head of 5.39 ft., and hence, no appreciable advantage as regards depth is obtained by using vertical stays at 4 ft. centres instead of a horizontal rail, while the cost would almost certainly be greater with the arrangement of Fig. 57 than with that of Fig. 48.

Perhaps a more definite comparison may be obtained as follows—

With the arrangement of Fig. 48, a $\frac{3}{8}$ in. plate thickness gave a permissible head of 7.06 ft., and for this head with the arrangement now under discussion we should have—

$$H = (7.06 \times 12) - \left(\frac{L}{2}\right).$$

Taking $\left(\frac{L}{2}\right)$ as 22.72 in. (obtained from a first approximation)—

$$H = 84.72 - 22.72 = 62 \text{ in.}$$

$$\therefore 933,454 \left(\frac{t}{L}\right)^2 = 62 \text{ in.}$$

Inserting $\frac{3}{8}$ in. for t , and transposing—

$$L^2 = \frac{933,454 \times 9}{62 \times 64} = \frac{8,401,086}{3968} = 2117,$$

$$\therefore L = \sqrt{2117} = 46 \text{ in.}$$

Now, the cost of the top curb with the arrangement of Fig. 57 would probably not be appreciably less than that of the horizontal rail for the arrangement of Fig. 48, and taking other factors as similar in both cases, the cost of the vertical stays at about 3 ft. 10 in. centres—or, at least, a large proportion of that cost—would appear to be outstanding to the detriment of the arrangement indicated in Fig. 57.

Another instance will, perhaps, suffice.

With the arrangement of Fig. 48, a wall of $\frac{1}{4}$ in. plate gave a permissible head of 5.39 ft., and for this head with the arrangement of Fig. 57 we should have—

$$H = (5.39 \times 12) - \left(\frac{L}{2}\right).$$

Taking $\frac{L}{2}$ as 17.68 (obtained from a first approximation)—

$$H = 64.68 - 17.68 = 47 \text{ in.}$$

$$\therefore 933,454 \left(\frac{t}{L}\right)^2 = 47 \text{ in.}$$

Inserting $\frac{1}{4}$ in. for t , and transposing—

$$L^2 = \frac{933,454}{16 \times 47} = \frac{933,454}{752} = 1241,$$

$$\therefore L = \sqrt{1241} = 35.2 \text{ in.}$$

Hence, for walls of $\frac{1}{4}$ in. plate, the cost of the vertical stays at about 3 ft. centres would appear to be outstanding to the disadvantage of the arrangement indicated in Fig. 57, as compared with that shown in Fig. 48.

39.—*Arrangement of Curbs, Rails, and Stiffeners.*—We may conclude our discussion of this class of walls for rectangular tanks with a few remarks upon the arrangement and fitting of curbs, rails, and stays, and the means for providing them with the necessary strength and stiffness to take up the pressures which may be applied to them. Also, as to the salient facts regarding the influence of the vertical stays upon the sheeting with respect to the continuity of the latter; and the assumptions regarding continuity which may properly be applied under given conditions.

Bottom angle-curbs are invariably placed inside the tank, and for obvious reasons this is the best arrangement. It is the usual practice to place top curbs on the outside of tank walls, and this again, from all practical points of view, is as it should be.

Vertical stays, however, should be placed on the outside of the tank walls, instead of inside as is practically always done. With the stay (or stiffener) inside the tank, as indicated at (a) in Fig. 59, the outward pressure of the contained liquid upon the wall sheeting must be transmitted to the stay by means of rivets or bolts, each one

of which must be made tight against leakage. This involves a large amount of drilling, riveting, and caulking—three items which should be minimised—as well as time and labour. With the stay placed outside the tank, as at (b) in Fig. 59, the wall sheeting is pressed against its support by the contained liquid, and only a few rivets are required to hold them in their proper relative positions.

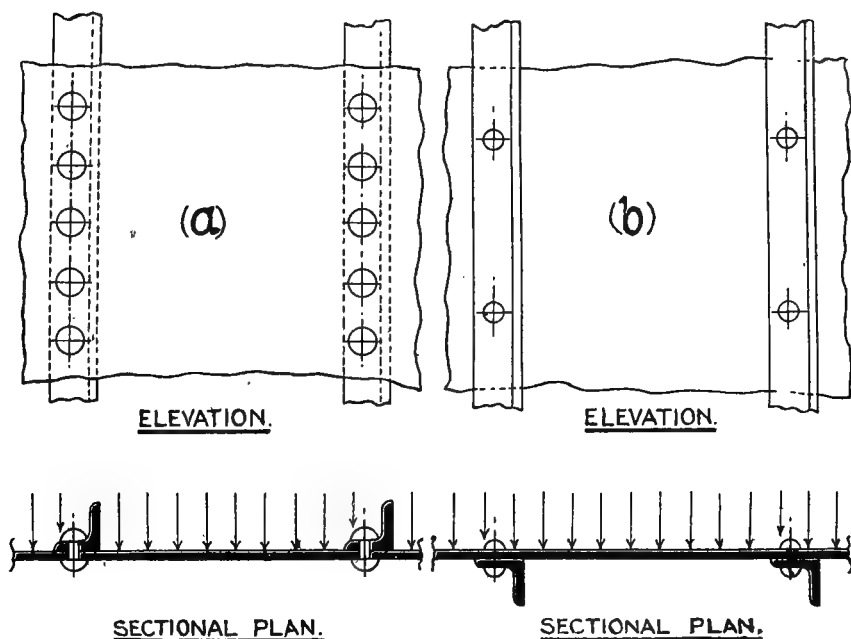


FIG. 59.

The sheeting itself is affected by this question, as well as the cost of the stiffeners. With sheeting continuous over the stiffeners, the maximum bending moment ($\frac{W L}{12}$) in the sheeting occurs at the stiffeners, and the full section of the beam strip at these sections has been assumed in the foregoing examples. Now, it is obvious that a considerable number of rivet-holes through the plate must appreciably reduce its strength, and that at the most severely

stressed sections. It is true that the rivets take up some of the bending action through their heads, but this is scarcely a factor to be relied upon, and the advantages of the arrangement shown at (b)—as compared with that shown at (a)—in Fig. 59, from this point of view, will be obvious. Tacking rivets, of small diameter, at, say, 12 in. pitch, should be quite sufficient to keep the sheeting always in contact with the stiffener for the purpose of preventing atmospheric moisture from finding its way between them and causing corrosion.

If it be objected that, since there are many tanks working satisfactorily with inside stiffeners, and with panel widths and liquid heads quite as large as those estimated in the foregoing examples for the plate thicknesses there stated, the weakening of the plate by the rivet-holes at the stiffeners must be negligible, there would still be a good reply. The weakening of the plate is undeniable, and the extent of the weakening cannot be otherwise than appreciable ; hence, either (or both) of two things must be happening in such tanks—(1) a greater intensity of stress than 7·5 tons per sq. in. in the sheeting is being withstood, or (2) the sheeting is not acting as a continuous beam in the manner assumed, but is transmitting the pressures in some other way.

Now, it has been stated above that there is a wide scope for careful and thorough investigation, by some competent and disinterested observer, into the question of tank sheeting, with the object of ascertaining, as precisely as may be, both the manner in which the pressures are actually transmitted by the sheeting, and the working stresses which may properly be permitted—or, at least, allowed for in calculations—in the material. As has already been suggested, it is quite possible that the sheeting does not act as a series of beam-strips at all in some types of tanks—particularly that illustrated in Fig. 57—and if this were so, it is obvious that a design based upon beam action at a stress of 7·5 tons per sq. in. could not (except by accident) be justified by fact.

In the absence of established knowledge, however, we can only proceed by means of logical argument based upon what seem to be the most reasonable probabilities ; and if the sheeting does act as a series of beam-strips, then, as we have shown, a working stress of 7·5 tons per sq. in. is the maximum which should be permitted

(in calculations) for British Standard mild steel as used for constructional purposes.

Moreover, the above objection would draw the reply that if a certain plate thickness is found sufficient in fact for a given panel width and liquid head with the vertical stiffeners arranged inside, as at (a) in Fig. 59, then it is practically certain that sheeting of some less thickness would be sufficient with the arrangement shown at (b).

Let it not be thought, from the foregoing admission, that the whole basis of design employed in these pages may be hopelessly false. It is only the method of staying indicated in Fig. 57 which is open to serious suspicion, and that for the reasons stated in the discussion of that particular method.

Horizontal supporting rails, such as that indicated in Fig. 48. should be placed on the outside of the tank walls, for the same reasons as those which apply to similar placing of vertical stiffeners.

CHAPTER V

FRAMING FOR RECTANGULAR TANKS

40. *Horizontal Ties*.—The best methods for providing horizontal rails or curbs with sufficient strength and stiffness for the proper transmission of the pressures which may be applied to them will depend upon, and should be governed by, the conditions and circumstances of individual cases.

In a tank of comparatively small width, horizontal ties may be used, connecting the curb or rail at one side with that at the other side, as indicated in Fig. 60, thus utilising the outward pressures at one side to counteract those at the other side.

If flat steel bars are used for these horizontal ties, they should be placed as at (a) in Fig. 60—*i. e.* with the greater cross-sectional dimension vertically—so that they may not sag unduly. As has been explained before, however, angle bars are preferable to flat bars for all such purposes, being at once more convenient to handle and fix, and also more effective in action. An angle bar tie should be placed as at (b) in Fig. 60—*i. e.* with its horizontal outstanding limb uppermost—to give as much stiffness as possible in the compression flange.

A convenient and effective method for attaching a horizontal transverse tie of angle bar to an external top curb is shown at (b) in Fig. 60, and at (c) in the same illustration is indicated an arrangement suitable for connecting the angle bar tie with an internal horizontal rail. Where the horizontal rail is placed on the outside of the tank wall, the tie may be connected with it by means of an internal cleat, riveted to the rail through the sheeting, as shown at (d) in Fig. 60.

If it be desired to use an angle bar tie of a section so slight in relation to its span that considerable sagging would be likely, stiffness may be obtained by means of light trussing, as indicated in

Fig. 61. As a rule, however, it is found that such work increases the cost of the tank much more than would the use of an angle bar

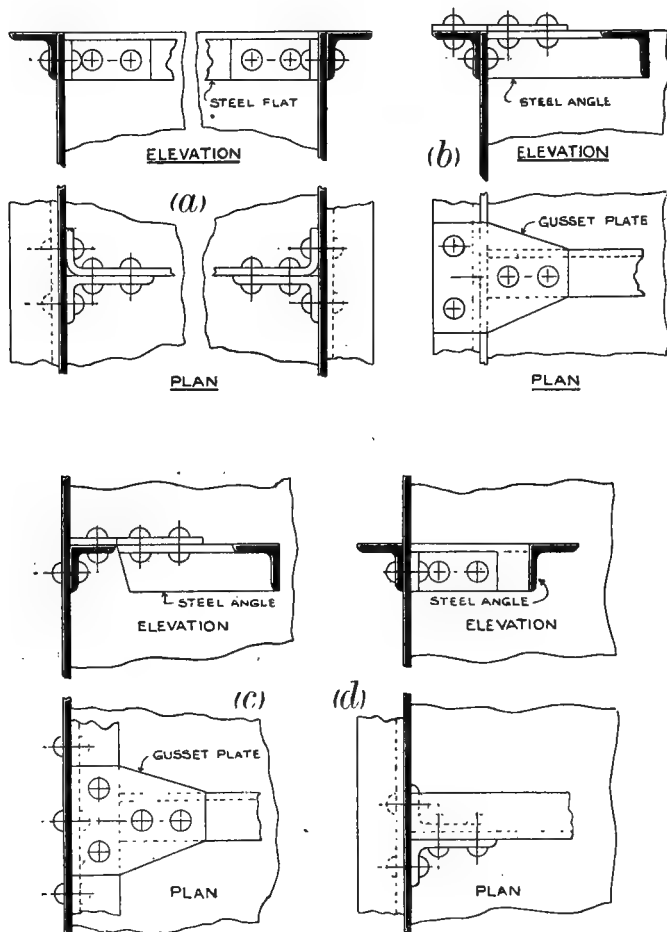


FIG. 60.

possessing sufficient strength and stiffness to properly cover the span, because of the labour involved in the trussing. Cases may—and do—arise in which the higher cost is less objectionable than the inconvenience which would result from an alternative method, and

for that reason the method should not be either condemned or ignored.

Horizontal transverse ties are sometimes carried across very wide tanks, sagging of the ties being prevented by means of posts or standards, erected on the floor of the tank, as indicated in

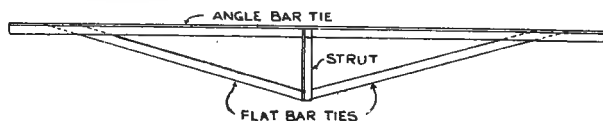


FIG. 61.

Fig. 62. There is no serious objection to urge against the use of such ties thus supported, though it is probable that, in general, better results might be obtained from somewhat less crude methods.

The posts or standards should, of course, be placed directly over a bearer in every case, and should be so cleated as to possess adequate stability for their purpose.

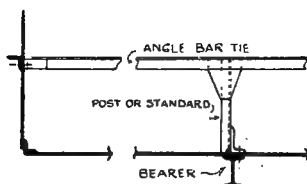


FIG 62.

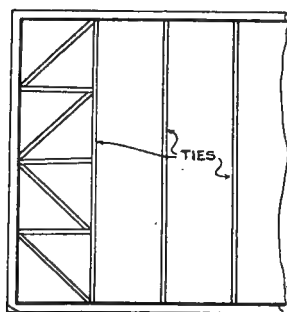
PLAN

FIG. 63.

It should be observed that the post or standard method of supporting very long ties has this advantage over the truss method of Fig. 61—that with the former the tie has simply to carry itself from standard to standard; whereas, with the latter, in addition to its having to carry itself from strut to support (probably about the same distance as from standard to standard with the method of Fig. 62), it must act as the compression boom of the truss, and will thus be called upon to bear a more or less considerable axial thrust,

which may call for bracing or support laterally in the horizontal plane as well.

With a very narrow tank the curbs or rails on the end walls may not need lateral support. Should such support be necessary, however, it may be provided satisfactorily by means of light framing to form a truss, using the nearest of the side-wall ties to form the compression boom, as indicated in Fig. 63.

The side-wall tie which forms the compression boom of this truss may be supported against flexure in the vertical plane by means of either a light trussing, similar to that indicated in Fig. 61; posts or standards, as in Fig. 62; or raking props from the

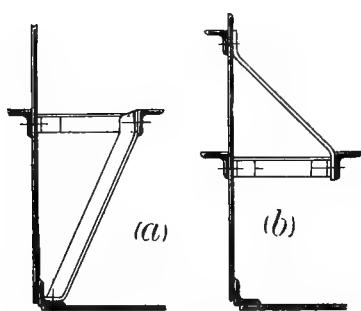


FIG. 64.

bottom curb, as shown at (a) in Fig. 64. Sometimes the weight of such truss framing is carried by means of slings from above, as indicated at (b) in Fig. 64. Where the rail to be stayed is a top curb, the sling support is, of course, impracticable; but even where the rail is so placed lower down the wall as to render suspension from above the easier way of supporting the inner boom, it is still preferable to use the

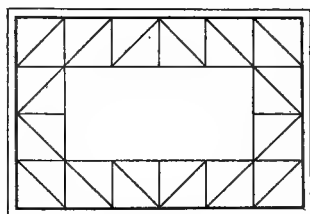
prop arrangement shown at (a) in Fig. 64. The bracket action set up with the latter will cause a restraining influence upon the wall; whereas the bracket action set up by the sling support from above causes an additional bending action on the wall, of the same sense as that due to the pressure of the contained liquid.

Horizontal transverse ties should be reasonably straight, carefully fitted, and so adjusted as to be properly up to their work. Slackness (due to careless fitting or adjustment) or kinks in such ties will in all probability be the means of permitting movement, with consequent uncertainty as to the distribution of loading and intensity of stress in the various pieces, as well as other effects which are highly undesirable in riveted plate work.

For the same reasons, the ties and their connections should be of such dimensions and proportions as will prevent any high intensity

of stress, either generally or locally, from the loading likely to be applied to them. Angle cleats should be sufficiently stiff to hold the ties without undue deformation, and not less than two rivets should be permitted for securing any piece, either to another member or to a cleat, gusset-plate, or other connecting piece.

41. *Trussed Framing*.—Where a tank is too wide to permit the efficient use of transverse horizontal ties from wall to wall, the horizontal curbs or rails may be provided with sufficient strength and stiffness by means of truss-framing, as indicated in Fig. 65. The rail or curb will form the tension boom of this truss, and must also be designed to properly withstand the local bending caused by its action as a beam providing lateral support for the sheeting between the panel points. The inner boom of the truss will, of course, be subjected to compression, and will almost certainly need support to prevent it from falling vertically. Such support may be provided by means of light trussing, as in Fig. 61; posts or standards, as in Fig. 62; or raking props, as at (a) in Fig. 64.



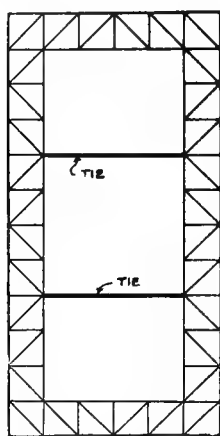
PLAN

FIG. 65.

Various modifications of this method, and combinations of truss-framing with horizontal transverse ties, will doubtless suggest themselves. A fairly obvious instance, for a tank of unusually great width and length, is indicated in Fig. 66. For such very large tanks, however, it is doubtful whether such methods are really economical. As will be shown presently, a better treatment for such cases may sometimes be obtained by using a framed tank, in which the wall framing is provided with sufficient stability to resist overturning, and thus needs no additional staying.

42. *Raking Stays*.—Raking stays, indicated in Fig. 67, should not be employed if any other method of supporting the walls is practicable. The vertical component of the axial load applied to the raking stay is applied as a downward load upon the curb or rail, and practically the whole of this downward load must obviously be taken by the wall sheeting as a compression—a highly undesirable state of affairs. At the foot of the stay, a similar load is applied

horizontally to the floor sheeting, unless the stay be secured to a substantial bearer lying in the vertical plane which contains the stay—and even then it is probable that transverse loading will be applied to some of the steel bearers, or other construction supporting the tank, which it is much better to avoid if possible. In framed tanks, raking stays are less objectionable, as the induced thrusts may be confined to regular constructional members, and thus be properly provided for. This point will be fully dealt with in due course.



PLAN

FIG. 66.

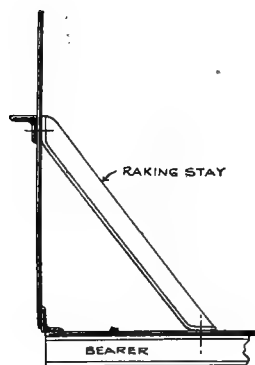


FIG. 67.

43. *Action of Curbs and Rails.*—Let us consider the conditions of loading under which a top curb or horizontal supporting rail for a tank wall must act.

It is often stated that a rectangular tank tends to assume a cylindrical form under the pressure of the contained liquid; and this, if interpreted broadly, is true. The tendency would be realised if the material of the containing walls were perfectly flexible—*i. e.* incapable of resisting bending actions while possessing adequate strength in tension. In ordinary tank work, however, this is not the case, and a somewhat closer examination of the facts is therefore necessary.

Consider the simple case of the top curb or horizontal supporting rail, without stays or bracing, for a tank square on plan, as indicated in Fig. 68. The tendency will be for the curb or rail to become distorted, and to take up some form such as that indicated by the dotted lines.

Obviously, with so much distortion as is shown in the sketch, and assuming the tank floor to retain its shape, there would be wrinkling in the wall sheeting to such a degree as could not be tolerated in an actual tank. It is necessary, therefore, that the curbs or rails be provided with adequate stiffness to prevent undue bulging of the walls.

The outward pressures upon each pair of opposite walls must

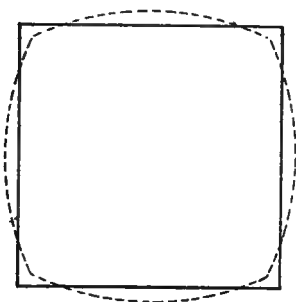


FIG. 68.

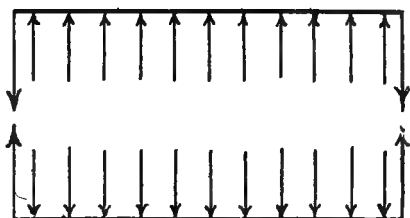


FIG. 69.

be taken up as a tension in the rails supporting the adjacent walls, this condition being as illustrated in Fig. 69. In addition, each rail must act as a beam in supporting its own wall against the outward pressures.

The dotted lines of Fig. 68 imply that there is no stiffness at the corners of the tank, that each wall has its own separate rail, and that the rails are connected at their adjacent ends by means of hinged joints.

If the rail or curb were of uniform (and, of course, adequate) strength and stiffness throughout—at all four corners of the tank, as well as along the walls—the tendency to opening at the corners would be resisted. The rail would then take up some form such as that indicated in Fig. 70, and the distortion would be less than with the rails hinged at the corners.

Clearly, the benefit of this continuity would be reduced if the tank were rectangular in plan instead of square, for the total pressures upon the shorter sides would not be sufficient to balance those acting upon the longer sides.

For the purposes of investigation, we may imagine the continuous rail to be cut at one of the corners, the rail straightened, appropriate supporting forces and restraining couples applied at the cut ends and corner points to reproduce the conditions before cutting, and loading applied equivalent in all respects to that set up by liquid pressure in the tank.

The conditions for a square tank would then be as indicated in Fig. 71, and those for a rectangular tank as in Fig. 72, the difference

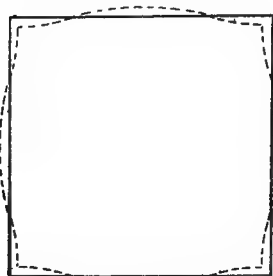


FIG. 70.

between the two sets of conditions, as regards flexural tendencies, being apparent.

In practice, it would be a comparatively simple matter to reproduce these conditions of continuity in the top curb. The lengths of the walls would probably always be so short as to obviate joints in the rails except at the corners of the tank, and these might be gusseted, as shown in Fig. 73. The sketch indicates a curb of angle bar, and this is by far the most usual form; it is, however, not necessary to limit the section to an angle bar, any convenient section (either rolled or built-up), suitable for taking up the loading, being applicable.

For the curb on each wall of a tank square on plan, therefore, the conditions of loading would be as indicated in Fig. 74, and the investigation for design presents little difficulty.

To avoid unnecessary complication in the work, let us suppose

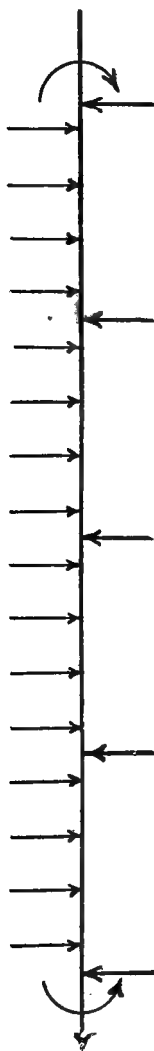


FIG. 71.

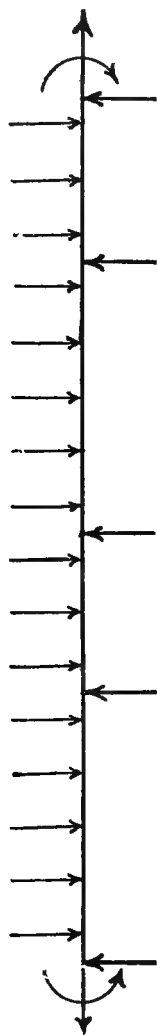


FIG. 72.

that the pressure applied to the curb by the sheeting has been determined, and let this pressure be taken as w lb. per foot run of curb. Then, if the length (and breadth) of the tank be l feet, the

total transverse load upon the curb will be $w l$ lb. At each end there will be a reaction equal to $\left(\frac{w l}{2}\right)$, supplied by the tensile resistance of the curb on the adjacent wall. In addition, there will be a couple at each end, of magnitude equal to $\left\{w l \left(\frac{l}{12}\right)\right\}$, and also a longitudinal tension equal to $\left(\frac{w l}{2}\right)$, applied by the curb on the adjacent wall.

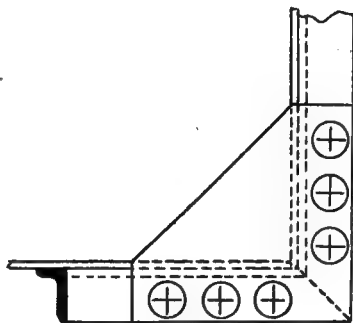


FIG. 73.

The maximum tensile stress in the curb section will thus be—

$$f = \frac{w l^2}{12 M} + \frac{w l}{2 A} = \frac{w l}{2} \left(\frac{A l + 6 M}{6 A M} \right),$$

where A is the area, and M the modulus, of the effective curb section.

Taking f as 16,800 lb. ($= 7.5$ tons) per sq. in. for mild steel, and transposing—

$$\left(\frac{A M}{A l + 6 M} \right) = \frac{w l}{201,600}$$

With any particular type of section, over a likely range, there is always a roughly approximate relation between A and M —*e. g.* with equal angles in the neighbourhood of 3 in. $\times$ 3 in. $\times$ $\frac{3}{8}$ in., $A = 2.5 M$; for larger angles up to 4 in. $\times$ 4 in. $\times$ $\frac{1}{2}$ in., $A = 2 M$; and so on,—and by making use of this fact, guiding values for A and M may be readily obtained.

Differences of opinion sometimes arise as to whether any—and,

if so, how much—of the wall sheeting may be regarded as forming part of the effective curb section in taking up the loading.

This question must, clearly, depend to a large extent upon the connections and riveting of the tank. It would be illogical, for instance, to calculate upon a certain sectional area of the sheeting, as forming part of the effective curb section, if the rivets securing the curb proper to the wall sheeting were not sufficient to develop the strength of more than a small fraction of that area. Nor would it be economical to ignore the wall sheeting entirely if the curb riveting were capable of developing the strength of a considerable area of it.

In ordinary circumstances, the wall sheeting in the neighbourhood of the top curb is very lightly stressed from the direct liquid pressure

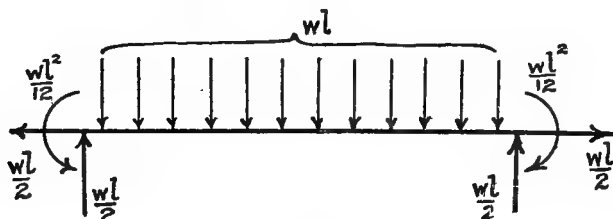


FIG. 74.

upon it, and hence it would seem a wise course to utilise as much of it as possible in building up the effective section of the curb. From the practical point of view, however, there may be other considerations which would turn an apparent economy into an actual extravagance.

For example, it might easily be that the cost of providing sufficient additional rivets to develop the strength of a desired area of the wall sheeting would far exceed the cost of the bare material which, by using a larger bar of the same type for the curb proper, would give the required strength. Moreover, the top curb is one of the few positions of an ordinary tank in which riveting may be minimised to meet the merest necessities of fastening. The bar is almost invariably placed on the outer surface of the sheeting, and there is little or no risk of leakage. Hence there are two opposing courses open—one, to obtain strength and stiffness in the curb

proper, and use as few rivets as practicable; and the other, to utilise a portion of the wall sheeting at the expense of additional riveting. Circumstances alone can determine which of these courses will be preferable in any given case, and it would be worse than useless to attempt to generalise on the question.

It is almost unnecessary to point out that there is a limit to the amount of the wall sheeting which can be incorporated in the effective curb section, even with sufficient riveting to develop its strength. Whatever total stress is taken up by such sheeting must be transmitted back to the main member, in which it will set up shearing forces, and only so much flange stress can be developed as the web will stand in shear. The conditions are exactly those of the ordinary built-up plate girder, and detailed discussion would therefore be out of place here.

With a tank rectangular on plan, it is doubtful whether the continuity of the top curb obtained by merely gusseting at the corners would be of any appreciable value. Much would depend of course, upon the relative lengths of the tank sides, but if the length of the tank were more than 1.5 times its breadth, the points of contraflexure in the longer span of curb would probably be so little removed from the ends that the maximum bending moment would occur at the centre of the span, and its magnitude might be but little less than for a similar span with the ends freely supported.

In such cases (and where the dimensions of the tank are not so large as to call for truss framing to the curbs) it is probable that transverse ties will give the most economical arrangement. A good plan is to space the transverse ties, so far as may be practicable, with the length of curb which each tie is to support approximately equal to the breadth of the tank, as indicated in Fig. 75. With gusseted corners the loading conditions for each panel of the curb would then be similar to those of Fig. 74.

Where a roof is to be provided over the tank, the transverse ties to the top curb may be used as part of the framing for supporting the roof. Methods for treating the ties in such cases will be suggested later.

Some doubt may be felt as to whether the gusseting at the tank corners will really give continuity of the curb, seeing that the latter will be simply mitre-butted, and its section considerably reduced

by rivet-holes in the portions where the bending action appears to be most effective.

It is, of course, true that one of the essential conditions of the continuous beam theory—viz., that the moment of inertia shall be constant—is violated, and that the reduction of section occurs where it is most objectionable according to that theory, but there are other factors which modify the circumstances of the case.

At the corners of the tank, each wall must necessarily be secured to the adjacent wall, and thus the sheeting will be very effectively supported along its vertical edge. One effect of this additional support will be to relieve the top curb of a considerable amount of the loading which would otherwise be applied to it in the neighbourhood of the gusseted corner connections.

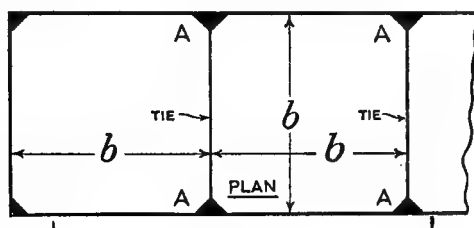


FIG. 75.

To attempt anything approaching a comprehensive investigation of the problem thus presented would involve more trouble than it would be worth—even supposing that an accurate solution were practicable—but a rational consideration of the case generally will show that there is not much likelihood of serious deficiency in a curb designed on the basis of Fig. 74 and the accompanying argument.

A similar question may arise with regard to those portions (marked A in Fig. 75) of the longer stretches of the top curb in a rectangular tank, where the transverse ties are connected to the curb. Here, however, the curb is actually continuous, and if the transverse tie connections be of the form illustrated at (a) in Fig. 60, there need be no reduction in the section of the curb beyond that involved in riveting the bar to the wall sheeting. Moreover, even though some other form of connection were employed—*e. g.* such as that shown at (b) in Fig. 60—causing a further reduction

of section, it is probable that a curb designed on the basis suggested above would be sufficient, provided that the amount of wall sheeting assumed as forming part of the effective curb section is not excessive. The bending moments—or, more correctly, the restraining moments—at the supports of a beam with “fixed” ends only reach the maximum magnitude of $\left(\frac{W L}{I_2}\right)$ on the assumption of point-reactions, and the bending-moment diagram shows that magnitude attained as a sharp cusp. A cleated or gusseted connection—such as those indicated in Fig. 60—will spread the reaction, and thus tend to ease matters for the curb, since the bending moment at the reduced section will be appreciably less than the $\left(\frac{W L}{I_2}\right)$, on which basis, it is suggested, the curb may be designed.

Clearly, if only the section of the actual bar were reckoned upon as forming the curb, without taking credit for any portion of the wall sheeting, one might feel confident that the additional strength due to the co-operation of the sheeting—which is not less real because ignored—would provide a sufficient margin to offset against the reduction in strength at the tie connections.

Altogether, therefore, it would seem that it is preferable, in ordinary circumstances, to take no account of the wall sheeting in designing the curb, thus leaving the assistance—unquestionably real though indeterminate—rendered by the sheeting to make good the shortcomings of the bar brought about through practical manipulation. After all, if the tank be economically designed, both as to proportions and sheeting thicknesses, a little weight more or less in the top curb is not likely to influence the total cost to any appreciable extent, while the advantage of having an assured sufficiency of strength and stiffness in the essential framing is beyond question.

Where the length of the tank is such that only one or two transverse ties are necessary, a slight lengthening in the ties—due to elastic strain in tension—would tend to reduce the bending moment at the panel points, and to increase it at the middle of the panels, thus rendering these moments less unequal. In any case, however, this would be of extremely slight effect—no more than a mere tendency, in fact,—while it would be quite inappreciable in cases

where the length of the tank is such that several transverse ties to the top curb are necessary.

As a rule it is not wise to allow rivets to bear direct tension, and for this reason the cleats of the tie connection shown at (a) in Fig. 60 might well be fastened to the curb by means of bolts, instead of rivets as shown. Obviously, much will depend upon the circumstances of a particular case in deciding whether bolts or rivets shall be used for this purpose. If the loading to be transmitted to the tie be of considerable magnitude, bolts will certainly be preferable; but if the tension be comparatively small, there need be no hesitation to use rivets. On the question of cost there is probably nothing to be gained either way; for, while bolts are almost certainly cheaper to fix than rivets in such cases, special marking is necessary to indicate the holes which are to be left unfilled for bolting, and the continuity of riveting is broken—both items adding to the cost of the work.

The design of the transverse ties, as such, does not call for any particular description. Adequate accommodation for rivets at the connections, and the provision of sufficient stiffness to prevent excessive sagging, are of at least equal importance with strength in tension; but the choice of a suitable section in any given case should not present much difficulty.

For connecting the tie to the curb it is well to notice that, with a gusset-plate—such as that shown at (b) in Fig. 60—the reduction in the curb section will usually depend more upon the size of the rivets used than upon their number. Hence, it is better, from this point of view, to use a larger number of small rivets than a smaller number of large rivets. As a rule, two rivets will be sufficient as regards resistance to shearing, but it may sometimes be advisable to use three (or even four) rivets of less diameter instead, particularly if there be but little margin of strength and stiffness in the curb.

Similar remarks apply also to the gusseted connections of the curb at the tank corners. Where the top curb is stiffened by means of trussed framing, as indicated in Figs. 63, 65, and 66, the ties and struts of the framing may be attached to the curb by means of cleats and gusset-plates. A typical detail for these connections is shown at (a) in Fig. 76. The connections on the inner (compression) boom of the truss framing may be as shown at (b) in Fig. 76,

which indicates also the attachment of the supporting props to the inner boom. In Fig. 64 the prop is shown attached to the inner face of the boom angle. This method is not infrequently used, but involves a somewhat complicated connection between the foot of the prop and the bottom curb angle. The arrangement indicated

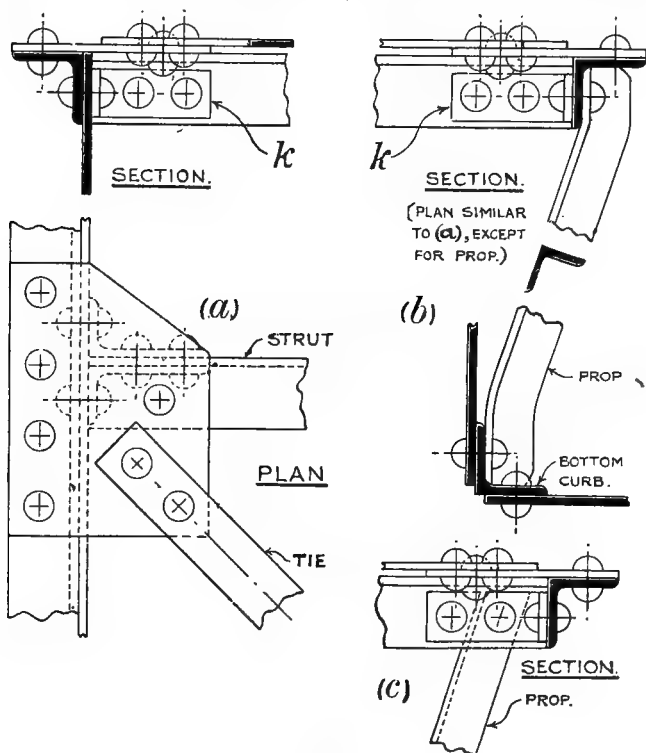


FIG. 76.

at (b) in Fig. 76 simplifies matters by turning the props so that the ordinary sheeting rivets through the bottom curb may be used to hold the feet of the props.

Another method for attaching the props to the framing is shown at (c) in Fig. 76. With the framing arranged as indicated in Figs. 63, 65, and 66, the shorter web members (lying at right angles with the booms) of the framing will be struts; they must, therefore, be

of angle (or other flanged) section, and the attachment of the props to them will do no harm if kept near the boom.

It is because these shorter web members are struts that the cleats marked k in Fig. 76 are necessary, to give proper means for transmitting the thrusts.

Provided that continuity be ensured, curbs and horizontal rails for rectangular tanks may be designed with a high degree of economy; moreover, as will be seen presently, there is a particular

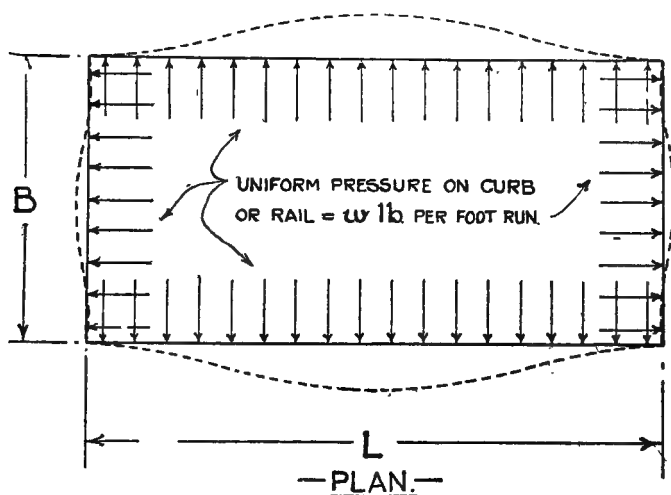


FIG. 77.

ratio of length to breadth which (under favourable circumstances) permits a more economical curb or rail than any other proportions.

It will be well to examine the conditions relating to such members, for cases frequently occur in which appreciable saving may be effected through taking proper account of these conditions.

Consider the case indicated in Fig. 77, the curb or rail being assumed truly "continuous" (*i.e.* having the moment of inertia of its cross-section constant) throughout. The shorter stretches of the curb, having less loading applied to them from the direct pressure of the liquid upon the walls which they support, will not be able to maintain the direction of their axes at the tank corners

unchanged in face of the preponderating moments applied by the longer stretches. The stretches of the curb or rail will deflect in some manner such as is indicated (with much exaggeration, of course) by the dotted lines in Fig. 77.

If the curb were cut at P (Fig. 78), a couple applied to each of the separated ends equal to the bending moment at P (represented by the symbol B_p), and the stretches rebated into a straight line, the conditions of loading and support would be as shown in Fig. 79. The bending moment diagram for these conditions could be drawn if the magnitude of the couple B_p were known, for it will consist of the four parabolic curves showing the bending moments for the four spans with their ends all freely supported, referred to a base-

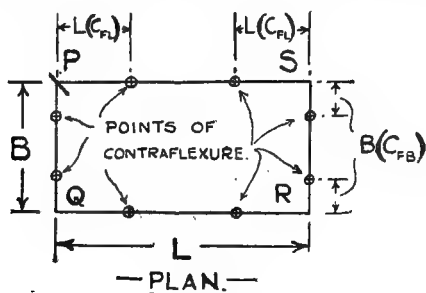


FIG. 78.

line B_p below the base-line of the parabolic curves, as shown at (a) in Fig. 79.

Now, the range between the sections T and V (Fig. 79) of this beam contains all the different stress conditions, the other ranges being mere repetitions. Hence, we may obtain all the necessary information from a consideration of this stretch. The loading to which the stretch T V is subjected comprises four distinct items—viz., (1) the couple B_B , which is the bending moment at the middle of the "breadth" stretches B ; (2) the couple B_L , which is the bending moment at the middle of the "length" stretches L ; (3) the uniform loading, w lb. per foot run, due to the direct pressure of the liquid upon the walls supported; and (4) the support (or reaction), $w\left(\frac{B+L}{2}\right)$, at Q.

The important point to notice is that the slope of the elastic line at T and V will be zero.

Regarding the end T as "built-in," the stretch T V may be treated as a cantilever, the slope at V being zero, and from this the magnitude of the couple B_L may be determined, for—

Slope at V =

$$\frac{d y}{d x} = \begin{cases} \frac{1}{6 E I} \left\{ w \left(\frac{B+L}{2} \right)^3 \right\} & \text{due to uniform loading;} \\ -\frac{1}{2 E I} \left\{ w \left(\frac{B+L}{2} \right) \left(\frac{B}{2} \right)^2 \right\} & \text{due to reaction at Q;} \\ -\frac{1}{E I} \left\{ B_L \left(\frac{B+L}{2} \right) \right\} & \text{due to couple } B_L. \end{cases}$$

$$\therefore \frac{d y}{d x} =$$

$$\frac{w}{48 E I} (B^3 + 3 B^2 L + 3 B L^2 + L^3 - 3 B^3 - 3 B^2 L) - \frac{1}{2 E I} \{ B_L (B + L) \}.$$

But this slope must be zero, and hence—

$$B_L = \frac{w (L^3 + 3 B L^2 - 2 B^3)}{24 (B + L)} = \frac{w (L^2 + 2 B L - 2 B^2)}{24}.$$

Therefore, from the diagram shown at (a) in Fig. 79—

$$B_F = \frac{w L^2}{8} - B_L = \frac{w}{24} (3 L^2 - L^2 - 2 B L + 2 B^2) = \frac{w}{12} (L^2 - B L + B^2).$$

Also—

$$\begin{aligned} B_B &= \frac{w B^2}{8} - B_F = \frac{w}{24} (3 B^2 - 2 L^2 + 2 B L - 2 B^2) \\ &= \frac{w}{24} (B^2 + 2 B L - 2 L^2). \end{aligned}$$

If $B = k L$, the expressions for these three moments become—

$$B_L = w L^2 \left(\frac{1 + 2k - 2k^2}{24} \right) = w L^2 (C_L);$$

$$B_F = w L^2 \left(\frac{1 - k + k^2}{12} \right) = w L^2 (C_F); \text{ and}$$

$$B_B = w L^2 \left(\frac{k^2 + 2k - 2}{24} \right) = w L^2 (\bar{C}_B).$$

By giving to k the values 0, 0.1, 0.2, . . . 0.9 and 1, the

corresponding values of C_L , C_P , and C_B may be calculated, as shown in the annexed table.

There is obviously no need to take values of k greater than 1,

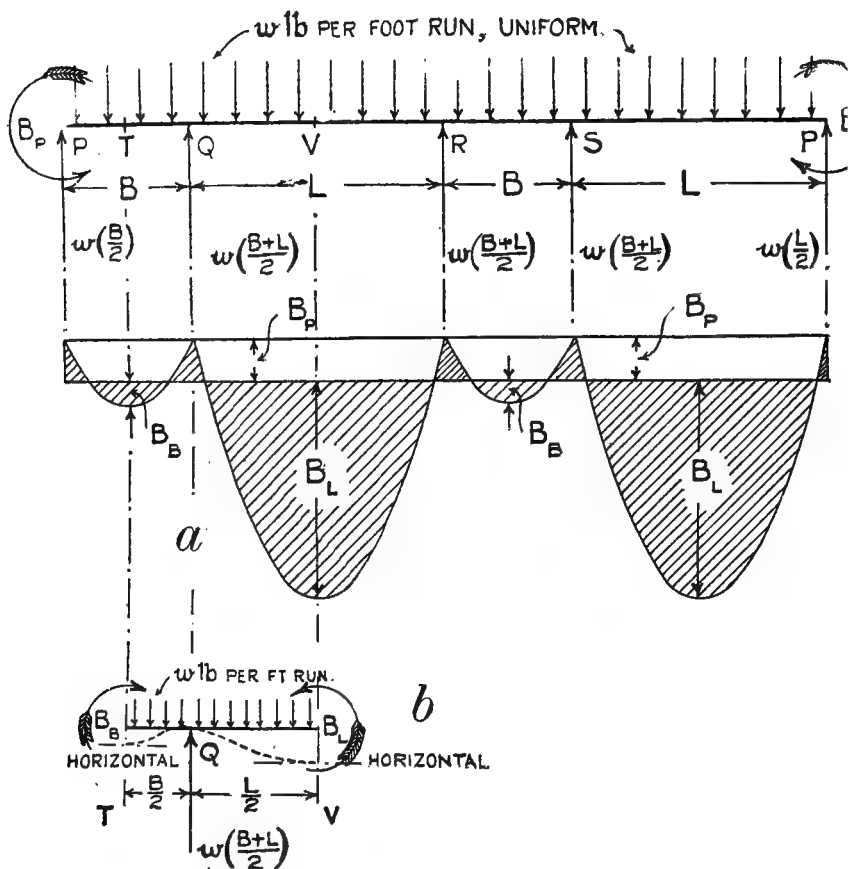


FIG. 79.

for the proportions thus represented would be mere repetitions of those tabulated, with L and B interchanged. If, therefore, the curb or rail be supposed of constant section throughout, and all bending moments be calculated with regard to L (the longer side), the

tabulated values cover the whole range of practical proportions for rectangular tanks.

Where k is very small, the case represented is that of an extremely narrow tank. The short stretches of curb would then transmit the end couples from one long stretch to the other, and the long stretches of curb would act as though built in to a rigid anchorage at each end. This is shown by the tabulated values of C_L , C_F , and C_B for $k = 0$, and the point is of more importance as tending to give a clear understanding of the facts than as likely to be of direct application in the practical design of tanks.

k .	C_L .	C_F .	C_B .	C_{FL} .	C_{FB} .
0.0	0.04167	— 0.08333	— 0.08333	0.211	—
0.1	0.04917	— 0.07583	— 0.07459	0.186	—
0.2	0.055	— 0.07	— 0.065	0.168	—
0.3	0.05917	— 0.06583	— 0.05459	0.156	—
0.4	0.06167	— 0.06333	— 0.04333	0.149	—
0.5	0.0625	— 0.0625	— 0.03125	0.146	—
0.6	0.06167	— 0.06333	— 0.01833	0.149	—
0.7	0.05917	— 0.06583	— 0.00458	0.156	—
0.8	0.055	— 0.07	+ 0.01	0.168	0.323
0.9	0.04917	— 0.07583	+ 0.02542	0.186	0.249
1.0	0.04167	— 0.08333	+ 0.04167	0.211	0.211

As the value of k increases from 0 to 0.5, the bending moment B_L increases from $\frac{w L^2}{24}$ to $\frac{w L^2}{16}$; the bending moment B_F decreases from $\frac{w L^2}{12}$ to $\frac{w L^2}{16}$; and the bending moment B_B decreases from $\frac{w L^2}{12}$ to $\frac{w L^2}{32}$, of the same kind or sense as B_F .

The bending-moment diagram for $k = 0.5$ is shown in Fig. 80.

As k increases from 0.5 to 1, B_L decreases from $\frac{w L^2}{16}$ to $\frac{w L^2}{24}$; B_F increases from $\frac{w L^2}{16}$ to $\frac{w L^2}{12}$; and B_B varies from $\frac{w L^2}{32}$ of the same sense as B_F to $\frac{w L^2}{24}$ of the same sense as B_L , passing a point of zero magnitude between $k = 0.7$ and $k = 0.8$.

Obviously, with $k = 1$ the tank is square on plan, and the bending-moment diagram for this case is shown in Fig. 81.

The value of k to give $B_B = 0$ may be determined by equating the expression for B_B with zero, whence—

$$k^2 + 2k - 2 = 0; \therefore k = \frac{-2 \pm \sqrt{4+8}}{2} = (\sqrt{3} - 1).$$

Therefore, for $B_B = 0$, the proportions are $B = L$ (0.7321).

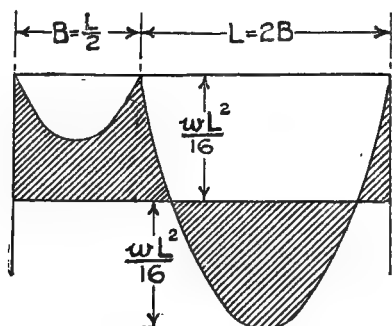


FIG. 80.

The points of contraflexure in the longer stretches (L) of the curb may be located for all values of k . In the shorter stretches (B) there will be no points of contraflexure unless k exceed $(\sqrt{3} - 1)$.

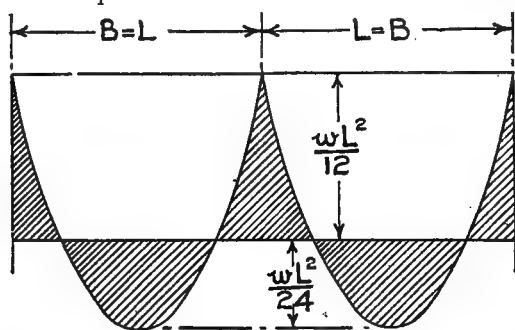


FIG. 81.

For the longer stretches—

$$\frac{w x^2}{2} = B_L; \text{ whence } x = L \sqrt{2 C_L},$$

where x is the distance between each point of contraflexure and

the centre of the span. For practical convenience it will be better to know the distance between the points of contraflexure and the tank corners; hence: $(0.5 L - x) = L (0.5 - \sqrt{2} C_L)$. This may be written as $L (C_{FL})$, and the values of (C_{FL}) corresponding to all

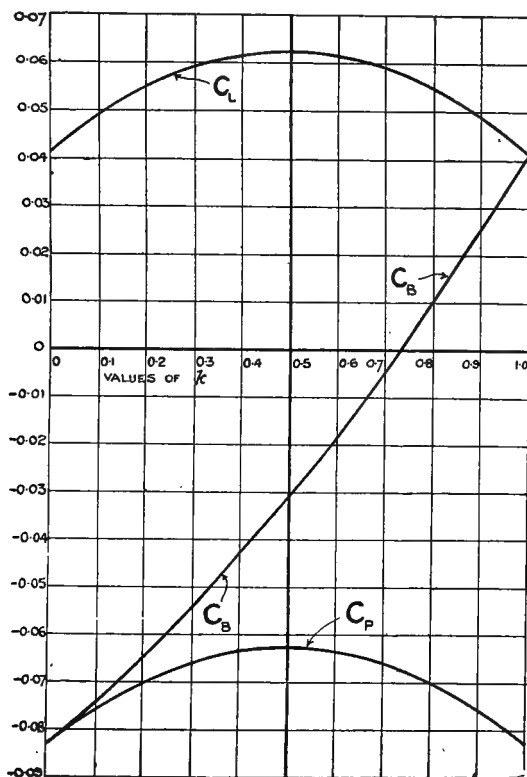


FIG. 82.

values of k , calculated from this relation, are given in the table with C_L , C_P , and C_B in page 151.

Similarly, for the shorter stretches, the distance between a tank corner and the nearer point of contraflexure will be equal to—

$$(0.5 B - L \sqrt{2} C_B) = B \left(0.5 - \frac{\sqrt{2} C_B}{k} \right).$$

This distance may be denoted as B (C_{FB}), and the values of (C_{FB}) corresponding to the possible values of k , calculated from this relation, are tabulated above with (C_{FL}).

The distances L (C_{FL}) and B (C_{FB}) are indicated in Fig. 78.

We shall now proceed to consider means by which the information deduced above may be turned to useful account in the practical design of tanks.

As they may be of assistance in actual designing, the values of C_L , C_P , and C_B , as tabulated above, are shown plotted with the

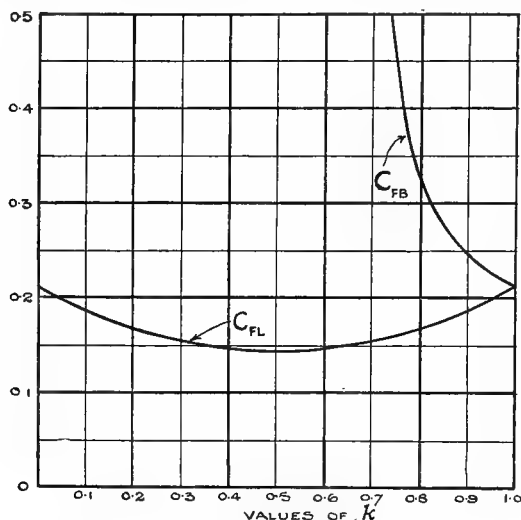


FIG. 83.

corresponding values of k , giving the curves of Fig. 82. The values of C_{FL} and C_{FB} are not shown in Fig. 82, as the diagram would be rendered inconveniently high by their inclusion plotted to the scale suitable for C_L , C_P , and C_B ; C_{FL} and C_{FB} are therefore shown plotted separately in Fig. 83 to a more convenient scale.

From Fig. 82 it will be clear that the proportions for rectangular tanks giving the least severe loading upon the curb or rail are in the neighbourhood of $L = 2 B$; though there is but 5.33% increase in the magnitude of the maximum bending moment (as compared with that for $B = 0.5 L$) where $B = 0.3 L$, or $B = 0.7 L$; and 12% increase where $B = 0.2 L$, or $B = 0.8 L$.

Now, as to the means for utilising, in practical design, the information obtained in the foregoing investigation.

The author would suggest that there are many cases in which the curb or rail might well be forged to a right-angle bend at each corner of the tank, and spliced at points of contraflexure. For instance, the curb or rail might be in four stretches, as indicated in Fig. 84, with splices at A, B, C, and D. The stretch A B would be similar to the stretch C D, and the stretch B C similar to the stretch D A, such similarity tending, of course, to reduce the costs of manufacture.

This method would not be suitable for heavy curbs, of channel or other large rolled sections; but, then, such sections are very seldom used for top curbs or horizontal rails in tank construction, and are not as a rule suitable for such purposes. A somewhat stronger objection is the increased difficulty in arranging for transport to avoid damaging the forged bars—though this is not likely to be anything like so troublesome with rectangular tanks of ordinary sizes and proportions as with the curved bars and plates for a cylindrical tank.

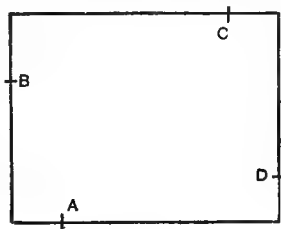


FIG. 84.

As regards cost of manufacture, the suggested method should compare favourably with that in common use; though in this connection much would, of course, depend upon the resources available concerning equipment and labour.

The splicing of the curb or rail at the points of contraflexure might well be effected by means of electric or other welding, for the wall sheeting will in all probability be sufficient to transmit the shearing force. It is, of course, quite possible that the curbs or rails might be in straight lengths, mitred and welded in position (by electrical or other means) at the tank corners; but one would require fairly exhaustive tests, and reasonably conclusive evidence, before recommending the placing of full reliance upon such methods at the corners of a tank curb, where a failure—or even a relatively small loss of strength or stiffness—might lead to disastrous consequences.

The attitude widely adopted towards the new forms of welding seems to the author unreasonable. It is similar to the attitude

adopted by so many engineers towards reinforced concrete, as well as to many other forms of construction which have been proposed. The question seems to be as to whether *all* connections shall be riveted, as formerly; or *all* welded—just as many engineers appear to have difficulty in deciding whether reinforced concrete shall be rigorously excluded, and structures built entirely of brickwork or steel as formerly; or whether all the old and tried methods of construction shall be swept away, leaving reinforced concrete to be employed for *all* structural work, regardless of its suitability—or otherwise.

The results of experience generally indicate that each method which can be justified on appeal to fact has its own particular field of usefulness; under certain types of circumstances a particular method is preferable to others, and under other circumstances its advantages are lost. And surely this is almost invariably the case with human beings, as well as with their productions. Why, then, should we demand, before admitting a suggested process, that it shall be capable of supplanting *all* others previously employed, and to show an improvement upon them all from every possible point of view?

There are many connections in tank work which could be readily made by welding; and there are many others for which riveting is—and will probably remain—best in every way. The sensible course would seem to be the application of the most suitable method for each operation, instead of being tied to one process for all. Of course, there are difficulties; but one so often finds that difficulties consist largely of inertia in those who meet them. The difficulties in the way of adopting new methods might frequently be more correctly defined as difficulties in the way of arousing sufficient interest on the part of those concerned.

If it be desired to arrange the curbs or rails in straight lengths, with riveted connections at the tank corners, the question arises as to what will be a suitable form of connection.

The ordinary gusset-plate, as indicated in Fig. 73, involves a more or less considerable reduction in the strength of the section, especially at the outermost rivet-holes for the gusset-plate, where none of the stiffness and strength of the latter has yet been added to the main member. Moreover, the vertical corner angle of the

tank would interfere with such a connection if used for a horizontal rail below the brim of the tank.

A preferable arrangement, suitable for curbs and rails alike, is the forged angle connection indicated in Fig. 85, or the cleated gusset shown in Fig. 86. The latter may be formed of a plate and angles, as at (a), or of a bent plate, as at (b), in Fig. 86.

The forged angle connection of Fig. 85 is particularly suitable for curbs or rails of angle section. The connecting piece should be of the same section as the curb or rail, and the parts secured to the main members should be of sufficient length (L) to accommodate the rivets (or—perhaps preferably—bolts) necessary to transmit all the loading from the main member to the connecting piece.

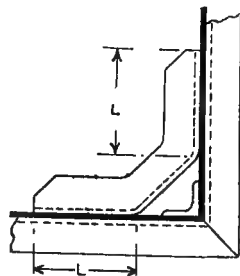


FIG. 85.

Similar provision is necessary with the angle cleats or plate flanges of the arrangement shown in Fig. 86. With this latter connection, the width of the gusset-plate required to accommodate the necessary rivets will give to the plate a considerably greater strength and stiffness to resist bending than that of the main curb

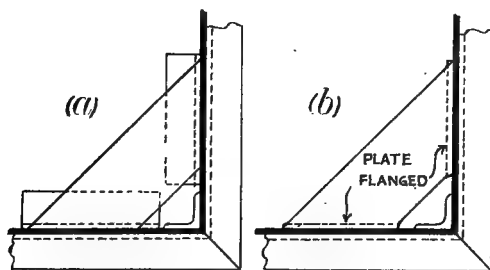


FIG. 86.

or rail; and this will tend to increase the bending moments at and near the connections.

It may be profitable to consider, very briefly, the effects of variation in the section of an otherwise "continuous" member upon its elastic line. Let us suppose that, if truly continuous, the stretch T V (see Fig. 79), under given loading, takes up the elastic

line indicated at (a) in Fig. 87. Now suppose a *reduction* in the section to occur over the support Q. The member will yield more than formerly under the bending action in (and near) the range in which the section is reduced; but since the contiguous portions must have a common slope at the point of contraflexure, the elastic line will be altered to some form such as that indicated at (b) in Fig. 87, the effect being to lengthen the (quasi) freely supported spans between the points of contraflexure in all four wall stretches of the curb, and thus to increase the bending moments B_L and B_B , while reducing the bending moment B_P .

If, on the other hand, the strength of the section were *increased* over the support, the member would yield less under the bending

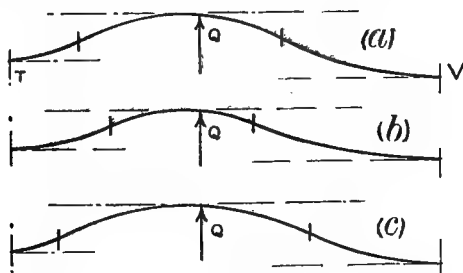


FIG. 87.

action in (and near) the range in which the section is increased; hence, the elastic line will be altered to some form such as that indicated at (c) in Fig. 87, the effect being to shorten the (quasi) freely supported spans between the points of contraflexure, and thus to decrease the bending moments B_L and B_B , while increasing the bending moment B_P .

Where the ratio borne by the length of the tank to its breadth is such that the bending moment in the curb or rail (as estimated on the basis described above) at the middle of the longer span is greater than that at the tank corners, the provision of greater stiffness and strength at the corner will tend to improve the conditions as regards loading. Conversely, if the tank be so proportioned that the maximum bending moment in the curb or rail, with true continuity, would occur at the tank corners, the provision of increased

stiffness at the corners might render the straining actions at those parts still more severe.

In seeking to obtain continuity for a top curb or horizontal rail by means of connections at the tank corners, much will depend—as regards the effective results—upon the “spread” of the con-

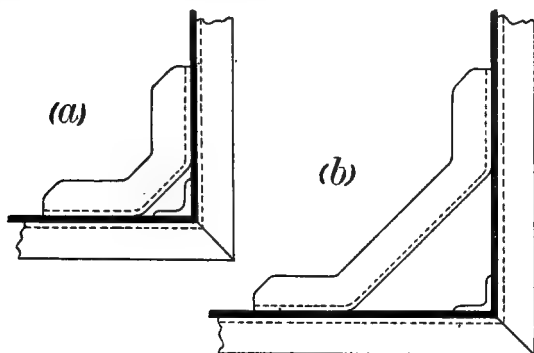


FIG. 88.

necting pieces across the corners. For example, with a forged angle connection of the type illustrated in Fig. 85, the conditions with the connecting piece close in to the corner, as indicated at (a) in Fig. 88, will be different from those with the connecting piece spread widely across the corner, as at (b) in Fig. 88.

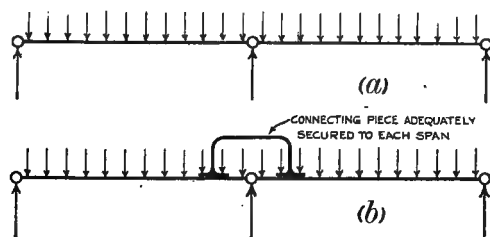


FIG. 89.

This point will, perhaps, be seen more clearly if the facts be considered from another point of view.

Suppose we have two contiguous freely supported spans, both uniformly loaded, and having a common support, as indicated at (a) in Fig. 89; and suppose that we seek to obtain continuity over

the common support by means of a connecting piece, as indicated at (b) in the same sketch. The similarity in effect between the conditions for this case and those for a curb or rail at a tank corner will be obvious.

It is important to notice that the bending moment in the connecting piece will be constant, whereas the bending moment in the corresponding range of a truly continuous beam would vary from section to section at a high rate.

The inferences to be drawn from this are of considerable practical importance, being capable of serving very useful purposes; but as they will doubtless be clearly apparent upon consideration, there seems to be no need for elaboration here. It will be found both interesting and instructive to investigate the elastic lines for curbs or rails connected in this manner, with different degrees of "spread" for the connecting pieces; and this is commended to the careful attention of the reader.

Two points should, however, be observed before passing on; viz.—

1. That, unless the tank be square on plan (or the curb stayed in equal panels on all walls), the bending moments in adjacent spans—assuming true continuity, and, of course, uniform loading throughout—are not symmetrical about the tank corners; and

2. That the rivets or bolts which secure the connecting pieces to the main members of the curb or rail should be capable of developing the strength of the connecting pieces, which will usually be more than that of the main members themselves.

It is probable that, in ordinary circumstances, the preferable course is to give the connecting pieces no more spread than will provide reasonable facility for the inspection, scaling, and painting of all surfaces, both of the connecting pieces and of the other constructional members in the vicinity. At the same time, however, instances may frequently occur in which an appreciable saving may be effected in the main curb or rail members through the exercise of a little care and thought in arranging the connecting pieces with a suitable spread; and such opportunities should certainly not be missed.

There is yet another method, which may be useful in many

cases, for advantageously connecting the curbs or rails at the corners of a rectangular tank. This method is indicated in Fig. 90, from which the underlying principles will be clear.

The arrangement shown at (a) is suitable for tanks square or nearly square on plan; and the arrangement at (b) for tanks in which the breadth is so small in comparison with the length that the shorter stretches of the curb or rail need no support other than that which can be afforded them at their ends by their attachment to the vertical corner angles of the tank.

Obviously, a combination of the two forms may prove convenient in some instances; and in others the arrangement shown at (b) in Fig. 90 may be used for both pairs of walls.

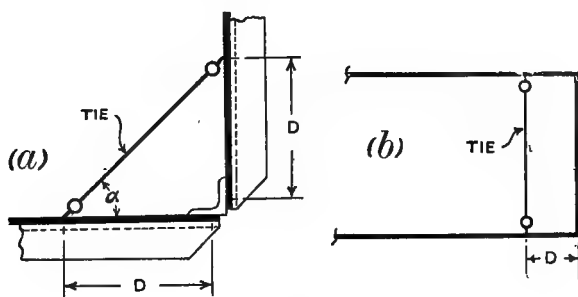


FIG. 90.

This method has a distinct advantage in that the curb or rail members need not be spliced, while the simply splayed ends may be made to present a very good appearance.

By suitably placing the ties in either arrangement, the ends of the curb or rail may be made to press lightly inwards against the vertical corner angles, thus reducing the more or less common tendency to leaking at the tank corners.

The ties are regarded as acting in tension only, and hence neither the ties nor their connections with the main members need be provided with stiffness to resist bending actions other than those due to the weight of the ties themselves.

A detail suitable for use with the arrangement shown at (a) in Fig. 90 is shown in Fig. 91, and it will be seen that no reduction in the strength of the main members is caused by this form of

connection. For the arrangement shown at (b) the details given in Fig. 60 for ordinary transverse ties will be suitable.

The conditions as regards loading in the curb or rail members will perhaps be clear from the following consideration—

With the arrangement shown at (a) in Fig. 90, suppose each of the ties to be replaced by a flexible but inextensible cord, passing

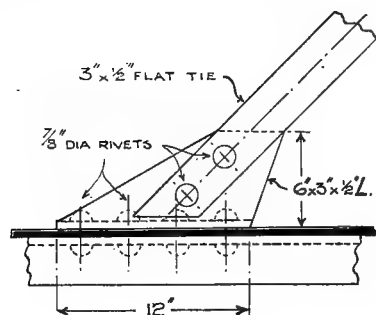


FIG. 91.

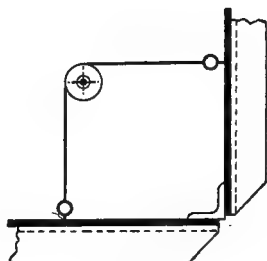


FIG. 92.

over a frictionless and adequately supported pulley, as indicated in Fig. 92. From this it is an easy step to the loading diagram of Fig. 93, and a simple basis for designs readily suggests itself. In the longer spans (and also in the shorter if the breadth of the tank be not much less than its length) there will be a point of zero slope



FIG. 93.

somewhere between the tie-connection and the tank corner—a section at which the conditions resemble those of a “built-in” end. The exact location of this section will depend upon the circumstances of each case—and, in particular, upon the ratio borne by the length of the tank to its breadth. A little consideration will show that all practical requirements should be satisfied if the members be designed for a maximum bending moment equal to

$\left(\frac{W L}{12}\right)$, where W is the total pressure acting upon the longer stretch, and L the full length of the tank. The same section should, of course, be used for the shorter as for the longer spans.

The tension in the tie may be estimated for designing by taking the bending moment $\left(\frac{W L}{12}\right)$, and dividing by D (Fig. 90), the distance between the tie-connection and the tank corner—taken rather on the small side for preference. The quotient will, of course, be the magnitude of each force of a couple having the arm D , and of magnitude equal to $\left(\frac{W L}{12}\right)$. Add to this force one-half of the total pressure (*i. e.* $\frac{W}{2}$) acting upon the longer spans of the curb or rail, and the resulting sum will be the total effective force normal to the longer wall of the tank, to be resisted by the tie. This total force, resolved to the angle of the tie, will give the load for which the tie and its connections should be designed, with any additional margin which may be considered desirable. Expressed symbolically, the tension in the tie will be—

$$\begin{aligned} T &= \left(\frac{W L}{12 D} + \frac{W}{2}\right) \operatorname{cosec} \alpha \\ &= W \operatorname{cosec} \alpha \left(\frac{L + 6 D}{12 D}\right). \end{aligned}$$

The additional tension in the main members between the ties, due to the inclination of the latter, should not be forgotten when employing the arrangement (*a*) of Fig. 90.

For the arrangement (*b*) the treatment follows simply and obviously from the foregoing discussion.

Convenient modifications of this method will doubtless present themselves, and as the object here is to be broadly suggestive rather than exhaustive, the matter may be left without further elaboration.

The application of the corner connections indicated in Figs. 85 and 86 to curbs and rails provided with transverse ties or trussing (or both) will be obvious; and it will also be clear that corner splicings may be rendered unnecessary with such curbs and rails

by suitably arranging the ties or trussing-panels on the lines indicated above in the discussion relating to the suggested method of Fig. 90.

Where two lengths of a curb or rail meet, the butt or mitre should be spliced with covers on all sides—even though such covers may not be necessary for purposes of strength—as a protection against corrosion. The advantage possessed by the arrangement of Fig. 90 in this respect is worthy of particular notice, as not only is the cost of the splittings saved, but all the surfaces are freely open for inspection, scaling, and painting.

It will probably have been noticed that, even with true continuity in a top curb or horizontal rail, if the tank be not square on plan (or the stretches stayed in equal panels on all walls) there will be a twisting action upon the vertical corner angles which no amount of gusseting or connection between the members can prevent. This twisting action is, of course, small with well-designed curbs and connections, but in many cases it is better avoided, since it provides an additional tendency to cause leaking by the opening of the seams between the corner angles and the wall sheeting. A small leak may soon become so large as to be serious; and a leak in a tank—particularly at a corner—once started, is often both troublesome and costly to remedy. The obvious course to avoid such twisting actions is to stay the curbs or rails in equal panels on all walls.

Where vertical stiffeners are used, they should (for the reasons given in pp. 127-130) be placed on the outside of the tank walls. Whether they be placed outside or in, however, the problem of their fitting and attachment to the top and bottom curbs is, apparently, much the same. If placed inside, according to the common arrangement shown in Fig. 57, each stiffener may be either joggled into the bottom curb or packed off the wall sheeting. If placed outside, as indicated at (b) in Fig. 59, it would seem that each stiffener may be either joggled into the top curb or packed off the wall sheeting.

No useful purpose would be served by entering into the old controversy as to the respective merits of joggling and packing. So much depends upon the equipment available (or, more commonly, lacking) in particular yards and shops, and so much upon personal

prejudice and inertia, that there seems little likelihood of an impartial comparison doing aught but raise a storm of protest.

There is, however, another method, which the author would suggest as being, perhaps, at once cheaper and more simple than either packing or joggling, while leaving no inaccessible pockets in which corrosion may go on undetected. This suggested method is shown in Fig. 94, and should need no further description. The dimensions given in the sketch should provide sufficient strength for all ordinary cases in which the use of vertical stiffeners can be reasonably justified, but there should be little difficulty in securing greater strength if desired. As has been stated already, the author is of opinion that vertical stiffeners—of this type, at least—should not be employed if any other method be available.

44. *Continuity of Sheeting.*—As regards continuity in the sheeting where vertical stiffeners are used on the outside of the walls with only light tack-riveting, it is probable that the most satisfactory results are to be obtained by using the thinnest sheeting practicable, spacing the stiffeners on the assumed basis of true continuity in the sheeting, and allowing only a half-width panel of sheeting at each end of every wall.

Where a vertical seam in the sheeting is necessary, it is the common practice to locate the seam at a stiffener, using for the latter a fairly broad-tabled tee (instead of an angle, as elsewhere) to form one cover for the butt of the sheeting, with a cover strip on the other side. This does not seem a proper thing to do at a section where the bending moment in the sheeting is apparently at its maximum—particularly as close riveting is essential to prevent leaking, and the section of the sheeting is considerably reduced in consequence. However, in view of the fact that trouble seldom arises from this cause, there would appear to be little ground for adverse criticism of the practice. The arguments given in pp. 157–159 regarding the probable behaviour of an otherwise continuous curb or rail weakened over a support, will, presumably, apply equally to sheeting; and hence it may be that the various portions

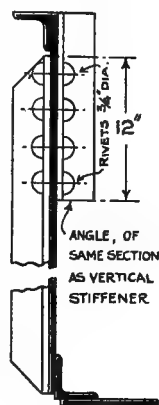


FIG. 94.

of the sheeting form among themselves some mutually satisfactory arrangement for sharing the load on a basis of give-and-take, working together amicably upon that basis. If so, it is to be regretted that their example is not more generally followed among their human acquaintance.

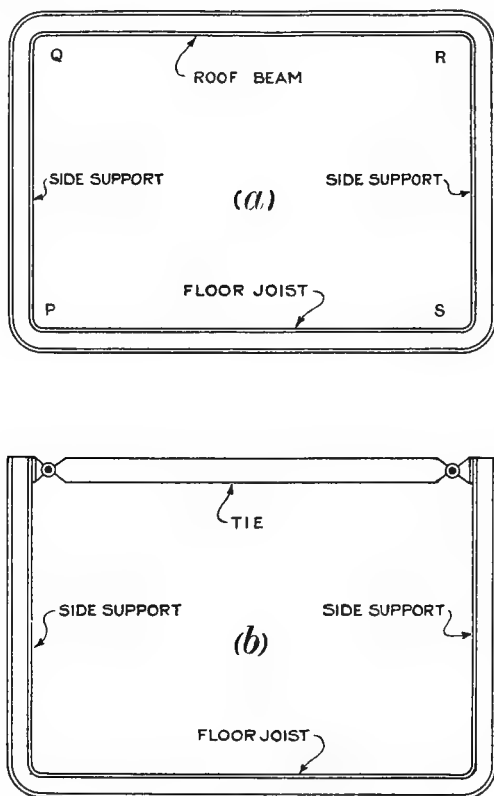


FIG. 95.

45. *Ribbed Tanks*.—In tanks of large depth the wall sheeting may be supported by vertical members continuous with the floor supports. The vertical supports may also have free support—by means of transverse ties or an adequate curb—at the brim or some lower level; or they may be made continuous with transverse beams carrying the roof. Two such cases are indicated in Fig. 95.

Now, it will be clear that unless the tank be kept full, so that the contained liquid is in contact with the underside of the roof, the assistance rendered to the side supports by a roof beam continuous with them cannot amount to much. Indeed, it is quite possible for a roof beam continuous with the side supports to increase—instead of diminishing—the loading upon the side supports. Even with the contained liquid subjected to a superimposed pressure, the conditions are not much improved, for the superimposed pressure will be transmitted to the side supports and floor beam by the contained liquid. The weight of the roof beam, and the roof construction and loading which it supports, cause the same kind of

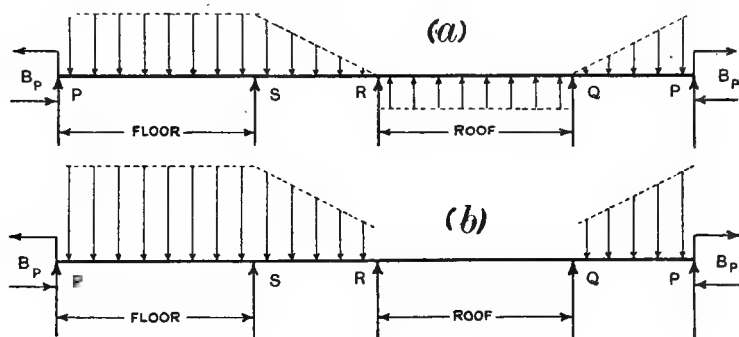


FIG. 96.

flexure in the continuous beam as does the outward pressure of the contained liquid acting upon the side supports, and this is the reason why such methods are not to be recommended for practical use.

Consider the supports indicated at (a) in Fig. 95; and imagine them cut at the corner P and rebated to form a straight continuous piece, as indicated at (a) in Fig. 96. With the contained liquid not quite (but very nearly) filling the tank, the loading on this straight member would be as indicated at (a) in Fig. 96, and the continuity couples at P are shown applied as external couples at each of the cut ends.

The weight supported by the roof beam Q R becomes an upward pressure in the straight piece of Fig. 96, and causes flexure of the same kind as does the direct loading upon the side walls.

Now suppose the tank to be full, and the contained liquid

subjected to a superimposed pressure of such magnitude that the upward pressure applied to the roof beam is just sufficient to nullify the weight of the beam and the roof (downward) loading which it supports. The loading upon the straight piece would then be as indicated at (b) in Fig. 96—the roof beam Q R free from loading, but the loading on the side supports and floor bearer correspondingly increased.

The stiffness of the roof stretch Q R would, of course, act as a restraint against further flexure of the side supports after the roof stretch had taken its flexure due to its own loading; but in that process the side supports would have been subjected to flexure by reason of their continuity with the roof stretch.

In the rare case of a tank in which the contained liquid is always subjected to a considerable superimposed pressure, there is more ground for making the roof beam continuous with the side supports; but the strongest reason for adopting the method in such a case is that it provides a ready means of meeting the necessity for tightness against leakage along the junctions of the side walls with the roof, and not so much because it promises advantage (as regards saving of material) from continuity effects.

For an ordinary storage tank a good deal will depend (as regards the assistance rendered to the side supports by a roof beam continuous with them) upon the proportions of the tank. Assuming true continuity in the frame (the section constant throughout all the four stretches), the side supports would receive much more assistance from the roof beam in a tank of small breadth and great depth than it would in a tank of great breadth and small depth. The more favourable case seldom occurs in practice, however; and it is doubtful whether the advantages to be gained are worth the trouble in any ordinary circumstances. It is probable that an effective tie, with ordinary connections, will generally be found preferable, from all points of view, to a roof beam continuous with the side supports.

With regard to continuity between the floor joist and the side supports, since the direct pressures upon these stretches tend to cause opposite kinds of flexure, it is obvious that continuity will be advantageous, provided that an effective and economical means for securing it be employed.

The most satisfactory way of obtaining such continuity (were it not for practical difficulties) would be by forming the side supports and floor joist of each "bent" in one piece; and this method has been used for tanks constructed of reinforced concrete. Rolled steel joists and channels, however, cannot be bent—even to a fairly flat curve—without a great deal of trouble and considerable loss of strength.

46. *Use of Raking Stays.*—From the discussion relating to continuity at tank corners for curbs and horizontal rails (pp. 157-159) it will be clear that little real success is likely to attend any effort to obtain continuity for the bent in separate stretches, by means of gussets, fishplates, and flange covers, for the section would

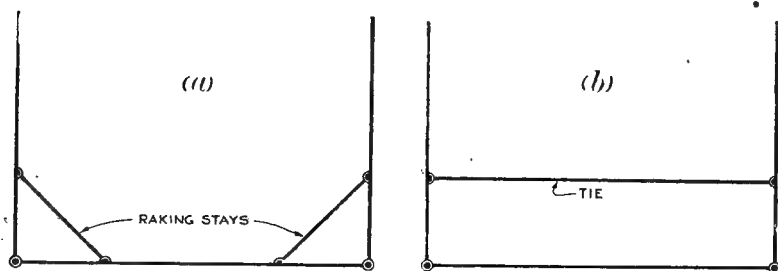


FIG. 97.

inevitably suffer more or less considerable reduction in parts due to the rivet holes.

Probably the best method, from the practical point of view, is to merely prevent horizontal movement at the feet of the vertical supports by cleating them adequately to the floor joists, and making no attempt to provide resistance to bending in the cleats. The overturning action on the side supports may then be taken up by raking stays secured to the floor joists, as indicated at (a) in Fig. 97; or by a transverse tie as at (b) in the same sketch.

It will be seen that the raking stays make for economy in one direction—the outward pressure of the contained liquid on the side supports is applied to the floor joist as an upward lifting tendency, and advantage may be taken of this in designing the floor joists and the steelwork or other construction supporting them. The arrangement shown at (b) in Fig. 97 does not give this additional economy;

but, on the other hand, this latter arrangement involves only one-half the number of connections necessary for the raking stays. Moreover, by reason of the inclination of the raking stays, the connections for the arrangement (*a*) will be subjected to more severe loading than those for the arrangement (*b*)—circumstances being similar for both, of course; and the bracket action of the raking stays in the arrangement (*a*) will set up compression in the lower ranges of the side supports. With reasonable care in designing, however, and under ordinary circumstances, these compressions need never assume such magnitudes as to require special provision in the members for their proper transmission.

The action of the raking stays will set up a tension in the portion of the floor joist between their feet, and a few remarks with regard to this are necessary, since the action may be turned to useful account.

The outward pulls being applied to the top flange of the joist, a bending action will be induced, tending to raise the middle portion and depress the ends—*i. e.*, a bending action of the opposite sense from that caused by the weight of the contained liquid. Owing to the fact that part of the tension will be taken by the floor plating, it is difficult (as well as inadvisable) to attempt anything approaching a definite estimate as to the magnitude of this reverse bending action, which may be generally reckoned upon as reducing the stresses in the floor joist due to the direct loading on the tank floor. The best course will be for the designer to treat each case upon its merits, and with a proper regard for its particular conditions and circumstances, remembering that while every legitimate endeavour should be made to prevent waste of any kind, excessive “cutting” of the floor bearers or side supports is likely to produce a leaky tank, through opening of the seams in the sheeting.

Where desirable, the effect of the reverse bending action upon the floor joist may be adjusted to harmonise with other factors; and this may be done by suitably placing the raking stays, and also by arrangement of the connections at their feet. The variations which may be obtained in the axial load taken by the raking stay will be seen presently, when we shall discuss the treatment for designing—including the most effective disposal of the raking stays. The influence of the connection we may investigate now.

Consider the detail of Fig. 98. The axial load F in the stay acts upon the cleats in the line $A F$; and were they not prevented from so doing, the cleats would move in the direction $A F$. Opposition to such motion is provided by the rivets S ; and, assuming that all the rivets participate equally in providing the resistance, both horizontally and vertically, the resultant resistance to vertical motion of the cleats will act in the line $B B$. At the point in which $B B$ intersects the bottom surface of the cleats, the horizontal resistance R_H and the vertical resistance R_V will be compounded to form the resultant resistance R , acting parallel with F . The connection is, therefore, subjected to an overturning action of magnitude

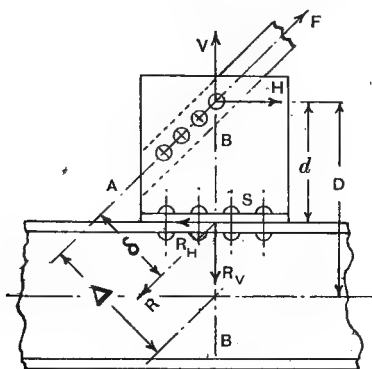


FIG. 98.

$(F @ \delta)$; and the reverse bending action applied to the floor beam is $(F @ \Delta)$.

Another way of regarding the matter is as follows. At the point in which $B B$ intersects $A F$, the force F will be resolved into its two components H and V , as indicated in Fig. 98. The overturning moment on the cleat, therefore, will be $(H @ d)$; and the reverse bending moment applied to the floor joist will be $(H @ D)$. Clearly, the result is the same with either line of argument.

It is scarcely necessary to point out that the rivets S must be capable of transmitting the overturning moment $(F @ \delta)$ to the floor beam, as well as the vertical and horizontal components of the force F .

Should it be desired to increase the overturning moment on the

cleats without altering the force F in either magnitude or position, it is only necessary to modify the cleats in such a manner that the rivets S may be rearranged so that the line $B B$ (in which their resultant resistance to vertical motion of the cleats may be supposed to act) shall intersect the line $A F$ higher up the rake of the stay. A contrary course will reduce the leverage D ; until, in the limit, the arrangement of Fig. 99 is obtained. In this case, δ being zero, the cleats are free from overturning moment; and the reverse bending moment applied to the floor joist is ($F @ D$), the leverage D being one-half the depth of the floor beam.

It will be obvious that this means of adjustment should be used very sparingly, and with the utmost discretion. It may be found useful in cases where the loading on the floor joist would otherwise

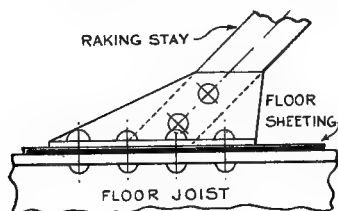


FIG. 99.

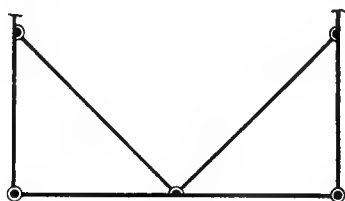


FIG. 100.

be such that some particularly convenient section falls short of the requirements by a small deficit.

Whatever proportion of the tension due to the raking stays may be decided upon, in a given case, as applicable to the floor joist, the estimated maximum stress in the joist should include the contributory stresses from all three sources of loading—viz., the weight of the liquid supported; the appropriated tension (regarded as spread uniformly over the cross-section); and the reverse bending action. The tension will affect only the portion of the joist between the feet of the raking stays; but the effect of the reverse bending action will extend beyond them, into the outer ranges.

For the sake of clearness, the floor sheeting is not shown in Fig. 98. Its effect upon the inferences deduced will, however, be obvious.

The total weight to be supported by the substructure will, of course, be unaffected by any reverse bending action in the floor

joists; but the incidence and distribution of the loading may be varied, and this is sometimes a matter of great convenience in practical arrangement.

In some cases the feet of the raking stays may be secured to a single pair of cleats for connection with the floor joist, as indicated in Fig. 100; and in such an arrangement neither the tension due to the raking stays, nor the otherwise consequent reverse bending action, will be applied to the floor joist.

Such connections should always be made by means of a pair of cleats—either of angle bar, or of bent plates if more convenient. The wringing action which is inevitable with a single cleat should be a sufficient argument against such connections, unless the force F be small—in which case it is probable that some other form of construction for the tank would be more efficient than that under discussion.

47. Action of Raking Stays.—

For depths not very great, a single raking stay, suitably placed, will provide all the support required, and then the top curb becomes unnecessary so far as regards the stability and stiffness of the wall framing.

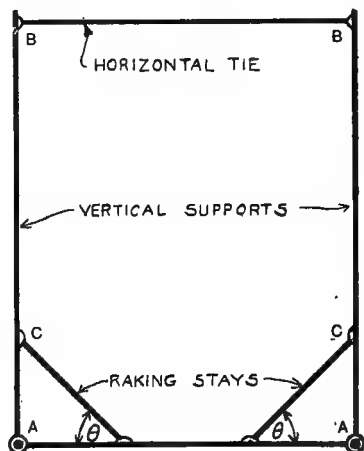


FIG. 101.

The treatment already given in the discussion concerning wall sheeting, supported solely by the bottom curb and a horizontal rail, will apply equally, in principle, to the vertical supports of the arrangement now under discussion; and the determination of the most effective position for the horizontal rail will serve also for the attachment of the raking stays to the vertical supports. Hence, the designs of the supports, stays, and connections for this case will be a simple matter needing no further elaboration here.

For the greatest depths likely to be required in ordinary tank work, provided the width be not very great, it is probable that a single raking stay in combination with a tie at the brim, as indicated

in Fig. 101, will provide the most economical and satisfactory arrangement.

In such circumstances the loading conditions with regard to the vertical support will be as indicated in Fig. 102; and the bending

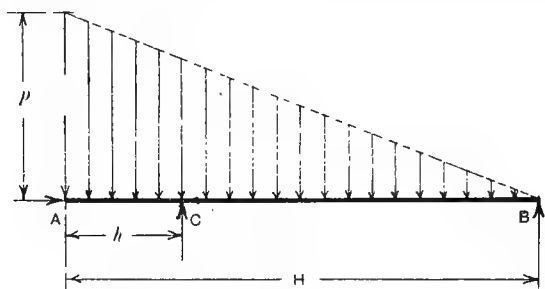


FIG. 102.

actions in the vertical support, as well as the loading to be transmitted by the raking and horizontal stays and their connections, may be estimated from a consideration of the elastic line of the vertical support, employing the usual assumptions.

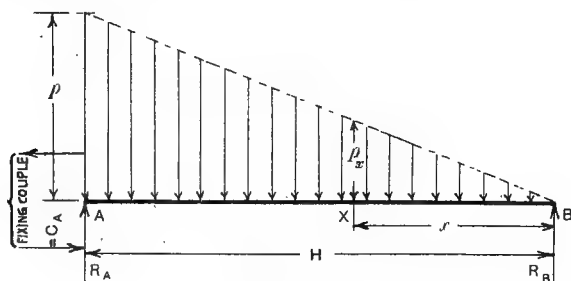


FIG. 103.

The points A, B, and C in Fig. 102 might reasonably be assumed to remain in line throughout the elastic deformations, and it is evident that, if h be anything less than (say) $\frac{H}{4}$, there will be no points of contraflexure between A and C, while a point of zero slope will occur somewhere about the middle of that range.

Clearly, then, if it be granted as a prerequisite that h shall be always about (or less than) $\frac{H}{4}$, we may assume that the conditions will be sensibly equivalent, as regards loading effects, to those of Fig. 103. The investigation will be simplified by the adoption of this basis, while any consequent error in the results will be relatively small, and on the side of safety.

Referring to Fig. 103, let p represent the maximum intensity of pressure per inch run of the vertical support. Then, if S be the distance between adjacent vertical supports, in inches; H the total height of the wall, in inches; and w the weight of the contained liquid per cubic inch;— $p = w H S$. The total pressure on the full range $A B$ will be $\left(\frac{p}{2}\right) H = \left(\frac{w H^2 S}{2}\right)$.

First consider the deflection of the piece $A B$ as a simple cantilever, subjected to the liquid pressures, fixed in position and direction at A , and without support at B .

At the section X , distant x from B , the loading intensity—

$$p_x = \left(\frac{x}{H}\right) p = \left(\frac{x}{H}\right) w H S = w x S;$$

and the total pressure on the range $X B$ —

$$= \left(\frac{w x S}{2}\right) x = \left(\frac{w x^2 S}{2}\right).$$

The bending moment at X will be—

$$B_x = \left(\frac{w x^2 S}{2}\right) \frac{x}{3} = \frac{w x^3 S}{6}.$$

But
$$\frac{d^2 y}{d x^2} = \frac{B}{E I};$$

and hence—
$$\frac{d^2 y}{d x^2} = \frac{w x^3 S}{6 E I} = \frac{w S}{6 E I} (x^3).$$

Integrating with respect to x —

$$\frac{d y}{d x} = \frac{w S}{6 E I} \left(\frac{x^4}{4} + C_1\right).$$

Since $\frac{d y}{d x} = 0$ when $x = H$, it follows that $C_1 = -\frac{H^4}{4}$; and hence—

$$\frac{d y}{d x} = \frac{w S}{6 E I} \left(\frac{x^4}{4} - \frac{H^4}{4} \right) = \frac{w S}{24 E I} (x^4 - H^4).$$

Integrating again with respect to x —

$$y = \frac{w S}{24 E I} \left(\frac{x^5}{5} - H^4 x + C_2 \right).$$

Since $y = 0$ when $x = H$, it follows that

$$C_2 = H^5 - \frac{H^5}{5} = \frac{4 H^5}{5};$$

and hence—

$$\begin{aligned} y &= \frac{w S}{24 E I} \left(\frac{x^5}{5} - H^4 x + \frac{4 H^5}{5} \right) \\ &= \frac{w S}{120 E I} (4 H^5 - 5 H^4 x + x^5). \end{aligned}$$

At B, where $x = 0$ —

$$y = \delta = \frac{w S}{120 E I} (4 H^5) = \frac{w S H^5}{30 E I}.$$

If the point B is to remain fixed as to position, the upward deflection which would be produced by a concentrated upward load at B equal in magnitude to the reaction R_B must be equal to that produced by the liquid pressures acting alone. Hence—

$$\frac{R_B H^3}{3 E I} = \frac{w S H^5}{30 E I};$$

whence—
$$R_B = \frac{w S H^2}{10}.$$

Thus, R_B is equal to one-fifth, and R_A to the remaining four-fifths, of the total liquid pressure on the vertical support A B.

The bending moment at X, therefore, will be—

$$\begin{aligned} B_X &= \frac{w x^3 S}{6} - R_B x = \frac{w x^3 S}{6} - \frac{w H^2 S x}{10} \\ &= \frac{w S}{30} (5 x^3 - 3 H^2 x). \end{aligned}$$

Differentiating with respect to x —

$$\frac{d B}{d x} = \frac{w S}{30} (15 x^2 - 3 H^2);$$

which gives the position of maximum negative bending moment;

for $\frac{d B}{d x} = 0$ when $(15 x^2 - 3 H^2) = 0$; *i. e.* when $15 x^2 = 3 H^2$;

whence $x^2 = \frac{H^2}{5}$; and—

$$x = H \sqrt{\frac{1}{5}} = \frac{H \sqrt{5}}{5} = 0.4472 H.$$

The magnitude of the maximum negative bending moment, therefore, will be—

$$\begin{aligned} B_{\max} &= \frac{w S}{30} \{ x (5 x^2 - 3 H^2) \} \\ &= \frac{w S}{30} \left\{ \frac{H \sqrt{5}}{5} (H^2 - 3 H^2) \right\} \\ &= -\frac{w S}{30} \left\{ \frac{H \sqrt{5}}{5} (2 H^2) \right\} \\ &= -\frac{w H^3 S \sqrt{5}}{75} = -0.0298 w H^3 S. \end{aligned}$$

The positive bending moment at A will be—

$$B_A = \frac{w S}{30} (5 H^3 - 3 H^3) = \frac{w H^3 S}{15} = +0.0667 w H^3 S;$$

and this, clearly, is the greatest bending moment in the vertical support.

Should the elastic deformations in the raking stay and its connections permit a slight change of slope at A, the bending moment at A will be reduced, and the negative bending moment will be increased. With ordinary care in design, manufacture, and fitting, however, such movement as is likely to occur in practice should reduce the maximum stresses in the vertical supports.

Assuming zero slope at A, there will be a point of contraflexure where $(5 x_0^3 - 3 H^2 x_0) = 0$; *i. e.* where $5 x_0^3 = 3 H^2 x_0$; whence $x_0^2 = \frac{3 H^2}{5}$, and—

$$x_0 = H \sqrt{\frac{3}{5}} = \frac{H \sqrt{15}}{5} = 0.7746 H.$$

Substituting $k H$ for x , the equation for the bending moment may be written—

$$B_x = \frac{w S}{30} (5 k^3 H^3 - 3 H^3 k) = \frac{w H^3 S}{30} (5 k^3 - 3 k) \\ = w H^3 S \left\{ \frac{k (5 k^2 - 3)}{30} \right\}.$$

This may be written as—

$$B_x = w H^3 S \{ Q \},$$

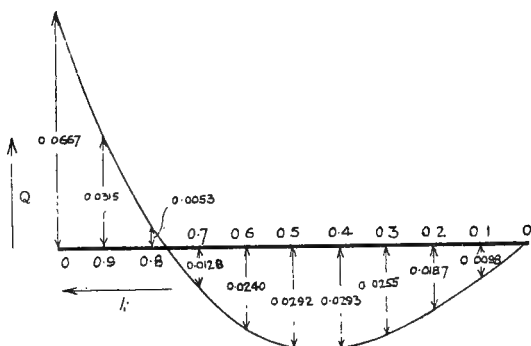


FIG. 104.

where $Q = \left\{ \frac{k (5 k^2 - 3)}{30} \right\}$; and giving to k the values 0, 0.1, 0.2, . . . 0.9, and 1.0, the corresponding values of the factor Q may be calculated, as in the accompanying table.

These corresponding values of k and Q may be plotted to give the diagram of Fig. 104, which will be found useful in practical design.

The raking stay and its connections should be designed to transmit a tensile load of—

$$T = \left(R_A + \frac{C_A}{h} \right) \sec. \theta,$$

where θ is the angle (see Fig. 101) between the raking stay and the floor joist.

k .	Q .	k .	Q .
0	0	0.6	- 0.0240
0.1	- 0.0098	0.7	- 0.0128
0.2	- 0.0187	0.8	+ 0.0053
0.3	- 0.0255	0.9	+ 0.0315
0.4	- 0.0293	1.0	+ 0.0667
0.5	- 0.0292		

Raking stays of the type under discussion should be inclined at 45° with the floor. At any greater angle the stay and its connections will be more severely loaded; and at any less angle the upward lifting action upon the floor joist will be reduced.

It should not be forgotten that the action of the raking stay will set up an additional compression in the portion of the vertical support marked A C in Figs. 101 and 102. As a rule this added compression will not form a serious item in the maximum stress in the vertical support; the wall sheeting may be relied upon to prevent sideway flexure in the support, and hence the latter may be designed for a permissible stress of 7.5 tons per sq. in. Cases may arise, however, in which provision for the additional compression is necessary or advisable; and for this reason its existence should not be ignored.

As regards the arrangement of the sheeting for this class of wall framing, it will be noticed that the vertical supports may be placed outside the sheeting, thus securing at once the most effective means of support for the sheeting and the minimum of necessary riveting; for the rivets securing the sheeting to its supports have no loading to transmit, and need therefore be only capable of holding the pieces in their proper relative positions and preventing the ingress of moisture and other corrosive agents between the sheeting and its supports. Such rivets may usually be $\frac{5}{8}$ in. diameter, and the pitch about (though it should not exceed) sixteen times the thickness of the sheeting.

The sheeting may usually be laid in horizontal strakes, a fairly light angle sufficing for the bottom curb to connect the wall and floor sheeting. Where the depth is sufficient to render such a proceeding worth while, the thickness of the sheeting may be reduced in the upper strakes. The strakes should be arranged alternately

“outer” and “inner” from the lowermost; and the horizontal seams may be single-riveted lap joints, with a packing strip inserted between the inner strakes and the vertical supports. Where the thickness of the sheeting is reduced, however, thin packing strips will be necessary even for the outer strakes above the first reduction in thickness, and for this, as well as for other reasons, it is open to question whether reduction of the plate thickness really pays in ordinary circumstances.

Where the length of the tank is so great that vertical seams are unavoidable, these seams should be single-riveted double-covered butt joints, and may be arranged to occur at a vertical support. For obvious reasons, the vertical seam in any strake should not occur immediately over that in the next strake below.

The sheeting may be designed as continuous over the vertical supports if these latter be spaced as described for ordinary vertical stiffeners supported by top and bottom curbs.

It has been assumed, so far, that the raking stays will lie in a vertical plane with the floor joist; but on one pair of opposite walls this will not be so. A suitable connection for the foot of the raking stay in such cases may be obtained by adequately riveting a short piece of joist (of the same section as the floor joists) to the floor plates and to the beams—if these latter lie conveniently; if not, the best plan is to insert trimmer joists between adjacent floor joists, to which they should be firmly cleated. The feet of the raking stays may then be secured to these trimmer joists through the floor plates.

Other methods of staying and supporting the side walls of rectangular tanks will doubtless suggest themselves; but as most of these will be either combinations or modifications of those suggested in this and the preceding chapters, there seems to be no need to particularise here with regard to them.

48. *Bottom Corner Connections.*—The arrangement of the bottom curb angles at the tank corners, and the junction of these with the vertical corner angles, sometimes causes unnecessary difficulty; and carelessness in this respect is a fruitful source of leaks at tank corners. A simple and sound rule is to make the bottom curb angle of thickness $\frac{1}{8}$ in. more than the wall or floor sheeting; while the width of the limbs need be no more than will suffice to accom-

moderate the necessary rivets for strength and for securing tightness against leaking. The vertical corner angle may conveniently be of the same section as the bottom curb; at least its thickness should be the same, though in some cases the width of its limbs may be slightly less.

One of the most important requirements is support for the meeting plates, so that the edges may be adequately caulked throughout their lengths; and this may be secured by mitring the curb angles, as indicated in Fig. 105, the vertical corner angle fitting closely against the top flange of the curb angle. The mitre of the curb angles may be well caulked, and the square edges of the vertical corner angle dressed down over the rounded edge of the curb angle, to prevent the ingress of water into these small spaces. The author is of opinion that this is an instance where electric or flame welding might be employed with advantage; it would certainly provide a more effective barrier against corrosion, and also greater stiffness for the caulking of the plate and angle edges. The only drawback is the cost—at present so high as to be prohibitive for most ordinary work—but perhaps that may become more rational in the future.

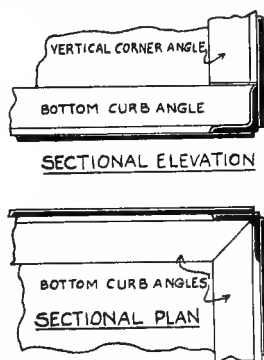


FIG. 105.

Before leaving the subject of the connection in which the vertical corner angle meets the bottom curbs of two adjacent walls in an ordinary rectangular tank, it may be well to notice a form which has been widely used for many years.

A three-way corner piece is formed, as indicated in Fig. 106, with the limbs of sections to match the corner angle and curbs. At one time, it is believed, these corner pieces were cast in steel; but latterly they have been much more commonly forged from the ordinary stock angle sections. The ends of the limbs are finished square, and are butted closely to the ends of the corner angle and curbs, each butt being covered on the inside with a round-backed cover strip of bent plate, accommodating three or more rivets on each side of the butt.

Clearly, it would be difficult and costly to cover the joints of the angles with the detail shown in Fig. 105, and in this respect the forged connecting piece of Fig. 106 has the advantage. In cases where electric or flame welding is not available, therefore, some such detail as that indicated in Fig. 106 may be preferable to that of Fig. 105; but the forging and fitting involved with the three-way corner piece are obviously expensive, and the butt strips cannot cover the joints of the angles completely.

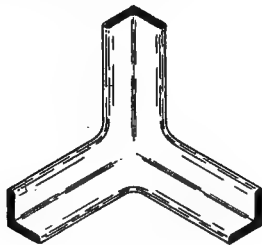


FIG. 106.

Another method which is sometimes (and perhaps frequently) used is to forge a corner connecting piece (two-way) for the bottom curbs only, butting and covering where the main curb angles meet the connecting piece. The vertical corner angle is then joggled to rest inside the angle of the corner piece, and the inevitable pocket spaces formed by the joggling along the top edges of the corner piece are "plugged"—which, being interpreted, usually means that they are more or less filled with some kind of putty or cement. This method cannot be recommended, for the cost of joggling and plugging, added to that of the forged two-way connecting piece, makes the total outlay but little (if at all) less than that for the three-way connecting piece of Fig. 106, while its disadvantages are obvious.

CHAPTER VI

TROUGH-BOTTOMED RECTANGULAR TANKS

49. *Action of a Trough Bottom.*—The question of trough bottoms for rectangular tanks has already been considered in Chapter II, so far as regards the effects of cross-sectional shape and proportions upon the economy of sheeting area for specified contents. We shall here discuss the trough-bottomed rectangular tank from the standpoint of practical design, both for strength and convenient arrangement.

First, let us examine the facts as regards loading for a typical case. In Fig. 107 the cross-section of such a tank is indicated, the trough being semi-elliptical, and its capacity about 25 per cent. more than that of the rectangular portion. The vertical side walls are assumed to act as longitudinal girders supporting the whole weight of the tank and its contents, they themselves being supported upon stanchions or other construction.

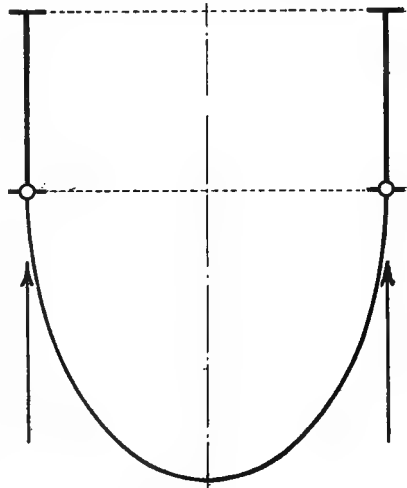


FIG. 107.

Confining our attention for the moment to the trough, let us imagine the skin composed of a number of links, sensibly rigid in themselves, but connected by means of frictionless hinges. We may then regard all the liquid pressures between the middle points of any contiguous pair of links as applied to the pivot connecting those links; and, obviously, the greater the number of links assumed,

the more nearly will the conditions approximate to those for a perfectly flexible trough.

With the dimensions indicated in Fig. 108, the length of the trough sheeting between the supports will be about 21 ft., and since both the trough and its loading may be regarded as symmetrical about the vertical centre line, we need only consider one-half of the trough. The length of skin for this being about 10.5 ft., we may (for the purposes of illustration) take the assumed links as about 2 ft. in length between pivot centres. The imaginary links are indicated in Fig. 108, and the depth of each pivot centre below the brim of the tank is figured. Taking 1 ft. run of the tank as a basis, the liquid pressures applied at the pivots may be calculated. Thus, if the depth of some particular pivot be D ft., the intensity of pressure at that level will be $W D$ lb. per sq. ft., where W is the weight of the contained liquid in lb. per cub. ft. With a fair number of assumed links, there will be but little error in regarding this intensity of pressure as constant between the middle points of each pair of adjacent links; and hence, the total liquid pressure at each pivot centre may be taken (in this case) as $W \times D \times 2 \text{ ft.} = 2 W D$ lb. per foot run of the tank. These pressures are indicated, acting in lines normal to the curve, in Fig. 108, and are lettered $A B, B C, \dots F G$, respectively. Setting out these forces to scale, the force polygon may be drawn as shown; and if each intersection be joined with the point h , the forces which must act at the pivots in order to maintain equilibrium may be determined.

To locate the point h we may argue that, since both the trough and its loading are symmetrical about the vertical centre line, the tensions $F H$ and $G H$ must be equal in magnitude and similarly inclined to the horizontal. Hence, if the force-line $f g$ be bisected in o , and a line (shown dotted in the diagram) drawn through o perpendicular to $f g$, the point h must lie in that line. Again, since the point in which the trough sheeting meets the lower flange of the vertical wall-girder is assumed to be pivoted, the uppermost link will swing outwards under the action of the liquid pressures. Supposing h to be vertically below b , a trial link-line may be drawn; and this was done in making the sketch for the illustration. This trial link-line (similar to the dotted link-line $A H, B H, \dots F H$ shown) was, however, found to be shorter than the added lengths

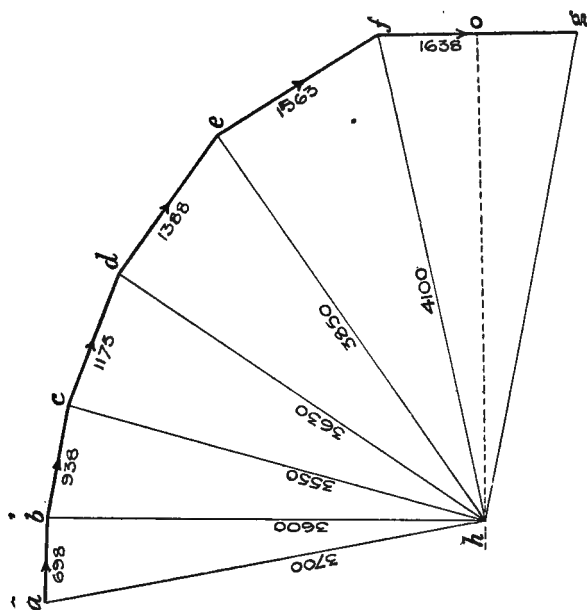
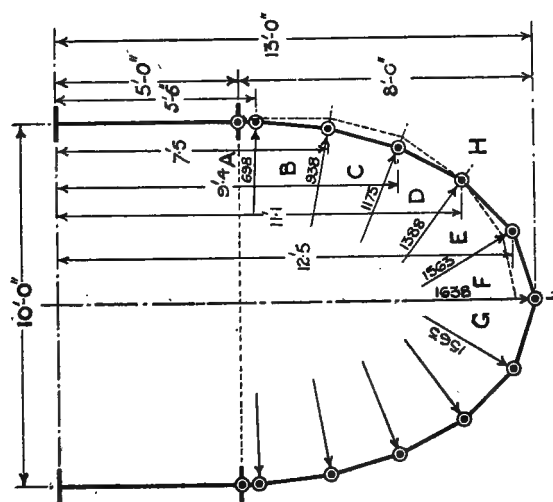


FIG. 108.



of the unloaded links; and since this obviously could not be in agreement with the facts, another position for the point h was obtained—slightly to the left of the first trial position. This gave the dotted link-line shown, the total length of which is sensibly equal to that of the unloaded links.

The point h could have been located by considering the liquid pressures upon a vertical plane containing the longitudinal axis of the tank, and thus determining the horizontal resistances, necessary for equilibrium, at the top and bottom of the trough. It does

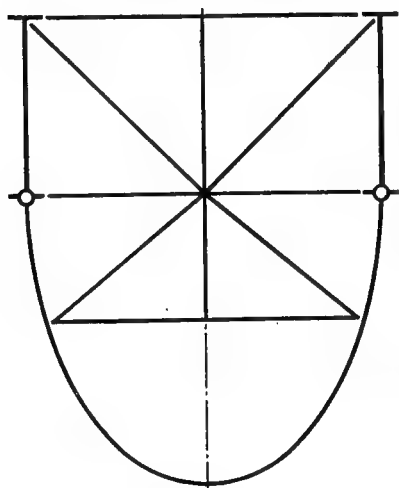


FIG. 109.

not follow, however, that such a location for h would satisfy the practical conditions any better than did the trial location described; for its determination would take no account of change in the shape of the trough under loading, and such changes must inevitably occur if the trough be flexible.

The reader is advised to construct for himself several diagrams such as that shown in Fig. 108, taking different proportions for the tank and trough, as well as different shapes (not forgetting the very convenient semi-cylindrical

form) for the trough, and examining for each the effects of variations in the surface level of the contained liquid. Some of these effects are described in pp. 57–60, Chapter II.

It will be obvious that a truly flexible trough would change its shape very considerably with alterations in the surface level; and it will also be obvious that circumferential seams—which are almost unavoidable with this form of construction—would not remain tight against leakage for long under such changes of shape, no matter how securely they might be made, nor how tightly caulked, in the first instance. Moreover, consideration of the tensions in the links, as indicated by the diagram, will show that no practicable

thickness for the trough sheeting could give sufficient strength and stiffness to maintain the shape of the trough against the loading; while any increase in the thickness of the trough sheeting for this purpose would reduce the advantages offered by such a form of construction.

The trough could, of course, be stiffened against deformation, either by means of curved ribs and bracing, as indicated in Fig. 109; or by rib plates held in position by horizontal transverse pieces, capable of acting as ties or struts according to the requirements of the loading, as shown in Fig. 110. Of these two methods, the latter is, perhaps, to be preferred for most cases, as being the more effective in preventing deformation. Both methods are, however, somewhat costly; and the advantages apparently obtainable from the use of a trough bottom (as against the ordinary flat bottom with its supporting floor construction) may be nullified if an elaborate system of stiffening and bracing be required. It must not be forgotten that the curving of the trough plating, besides being in itself somewhat expensive, adds appreciably to the costs of manufacture, transport, and erection; and a saving in mere weight may be a totally false basis for comparison as regards cost.

The form of the structure, as indicated in Fig. 107, is, however, unquestionably attractive for many reasons, and in the following pages we shall proceed to a further examination of the question, with a view to ascertaining whether some of the difficulties may be overcome by simple and practicable means, without unduly sacrificing the real and important advantages.

Much valuable information may be obtained, with very little trouble and at a cost of only a few pence, by arranging a simple model of the structure. The trough may be conveniently made from a strip of tracing cloth slung from two stiff laths, and fine sand may be used to represent the contained liquid. The trough may be

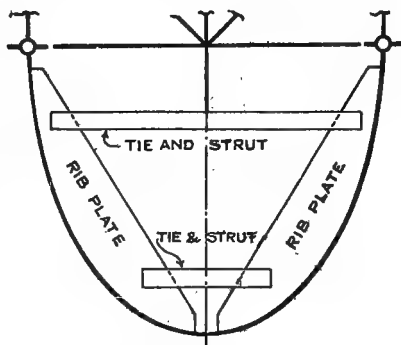


FIG. 110.

suspended between two vertical sheets to form the end bulkheads, one sheet being of glass or other transparent material, with horizontal and vertical lines scribed upon it at convenient intervals, so that distortions of the trough under loading may be readily observed and measured.

Since the sand will have an angle of repose, the conditions will not be exactly like those for a liquid; but then, as is well known, it is almost impossible to reproduce accurately in a model of any kind all the conditions of an actual structure. However, the information obtainable by such means is undeniably valuable, provided it be properly interpreted.

With a little more trouble and elaboration of the apparatus, more precise information may be obtained, but it is sufficient to give here the rough idea alone.

By making the trough of sufficient length, the effects of bracing and rib-plates may be observed. It will be found convenient to make the ribs and rib-plates of thin wood, and to fit them *outside* the trough. A trough composed of two strips of cloth, connected by a plain lap seam (without any adhesive) "riveted" with small paper fasteners, will soon show the effects likely to occur in the circumferential seams of a flexible trough.

The suggested apparatus is indicated in Fig. 111.

50. *Construction of Trough Bottoms.*—It will be obvious that if rib-plates or stiffeners be introduced to prevent distortion of the curved trough, the sheeting may be assumed to act as a series of beams between the supports. There is no particular difficulty in designing for such assumptions, of course, but much of the advantage offered by the trough form of construction may be lost, while there will be the additional cost for bending the sheeting and supports.

There is a promising field for research in this matter; for it is quite possible that, by locating the levels at which there is likely to be but little bending action in the trough, and arranging the trough sheeting in horizontal strakes with the longitudinal seams at these levels, the sheeting between the seams might not be seriously harmed through the changes of shape—particularly if the surface level of the contained liquid be not subject to wide and rapid variations.

Again, it is possible that, with the sheeting arranged in strips of (say) 5 ft. width, extending from the tank-wall girders on each

side to the invert of the trough, it might be found sufficient to provide adequate rib-plates (after the manner of Fig. 110) at each circumferential seam, with a stout rib (say of bulb angle section) along the longitudinal seam at the invert, between the rib-plates. Indeed, it is possible that if the trough be carefully shaped to suit the loading, so that bending actions would be small at full load, the plates might be capable of dealing with lesser loading without stiffeners or framing of any kind.

Experiments are necessary, on a reasonable scale, before any definite course for design could properly and profitably be laid down. It is useless to start theorising or calculating until more or less precise information has been obtained as to the manner in which the sheeting will act under variations of loading; and it is obvious that there are several factors—as, for example, the effects of the vertical bulkheads at the ends upon the curved trough sheeting—which may exercise a powerful (and, perhaps, even a determining) influence upon the resultant action under different sets of conditions.

From a consideration of the stresses as estimated in the diagram of Fig. 108, it seems likely that such research, if conducted upon right and broad-minded lines, might yield highly profitable results.

There would, however, appear to be a method available by which the principal advantages of the trough bottom might be secured without much uncertainty as to the manner in which the trough sheeting will act. The author has no knowledge of the method having been tried in an actual tank, but believes that it might give efficient and economical construction in favourable circumstances, and therefore offers the suggestions for consideration.

Primarily, this method consists of the use of flat plates for the main portions of the trough, supported and stayed against the pressures of the contained liquid, and arranged to give a section approximating more or less to the semi-elliptical shape which would provide the required storage capacity. The invert of the trough (which should possess ample stiffness) may be formed of a curved plate, bent to some convenient radius, the width being variable to suit the side sheeting of the trough.

To show the application of the method, let us consider the section indicated in Fig. 108, and treat it on this basis.

The cross-section might be as shown in Fig. 112, which will be found to give a capacity practically equal to that of Fig. 108. Using flat plates about 5 ft. in width, laid in horizontal strakes, there will be longitudinal seams at A and B. These may be single-riveted lap joints for all ordinary cases. Unless the tank be more than (say) 30 ft. in length, each strake may be in one piece. If the length be so great as to render the use of single-piece strakes either

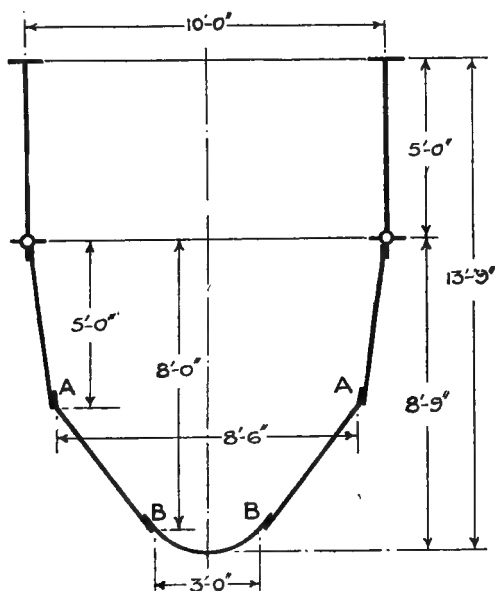


FIG. 112.

impracticable or inconvenient, circumferential seams will be necessary; and these may occur at stiffeners, the strakes "breaking joint" as usual.

Since the strakes will have different inclinations, it will be necessary to flange the plates for the longitudinal seams; but the flanging may be confined to one plate at each seam, as indicated in Fig. 112. By arranging the lowermost strake so that it shall be tangential to the curved invert plate, and leaving flat strips along the edges of the latter, flanging for the invert seams may be rendered unnecessary.

The supports or stiffeners for the sheeting may be of angle-bars,

with stays (also of angle-bars) cleated to them, as indicated in Fig. 113. By bending the supports as shown, packing strips or other adjustment to allow for the lap joints in the sheeting will become unnecessary. The small pocket spaces marked P may be plugged, or (preferably) filled with carefully fitted taper packing pieces. If the supports be placed outside the trough, they may be riveted to

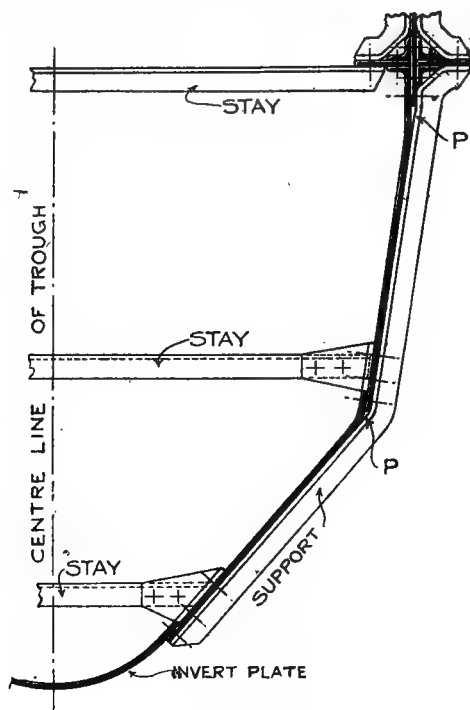


FIG. 113.

the sheeting by means of $\frac{5}{8}$ in.-diameter rivets, with a pitch equal to about sixteen times the thickness of the sheeting; if they be placed inside the trough, the riveting will need to be much stronger and closer, to transmit the liquid pressures to the supports and to maintain tightness against leakage.

It will be found that sheeting of sufficient thickness to transmit the liquid pressures to its supports by beam action (if the supports

be spaced at reasonable distances apart) will provide ample strength to support the weight of the entire trough and its contents by suspension; and that the lap seams, with ordinary riveting for tightness against leakage, will be subjected to very light shearing stresses. Since the tension in the sheeting due to the suspension of the trough and contained liquid will be at right angles to the stresses due to

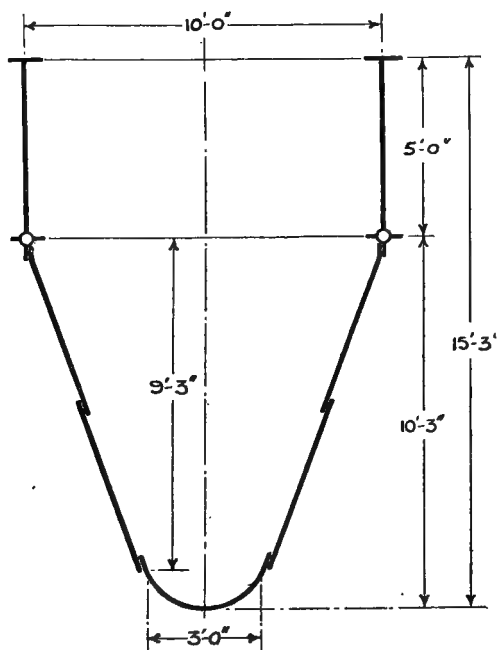


FIG. 114.

the beam action, the separate stresses need not be added together in estimating the working stresses.

This method should prove simple and cheap in erection; for the vertical tank-wall girders could be manufactured complete, in convenient lengths, before delivery to the site. They could be lifted into position and secured; and the trough framing, fitted and fixed in place, would provide practically all the support necessary for staging to erect and rivet the trough sheeting.

Obviously, the thickness of the trough sheeting may be varied

from strake to strake if desired; and economies could, doubtless, be effected in some cases by this means.

There would seem to be no need for consideration of the design and calculations in detail here, for all the loading and stresses may be estimated readily by straightforward methods of reasoning on the lines previously indicated. As already stated, the invert plate should be fairly stiff (in all probability at least one service pipe will be suspended from it); and the horizontal stays to the supports

should be capable of acting as struts to resist such thrusts as are likely to be applied to them.

As an alternative course, the whole of the trough sheeting (excepting the invert) might be flat, as indicated in Fig. 114, the invert being formed either of a curved plate (as shown in the sketch), a flat plate flanged along both edges, or a steel trough section, such as is sometimes used for the decks of bridges and the floors over subways. The relative advantages and disadvantages of such a method, as compared with that of Fig. 112, will be obvious.

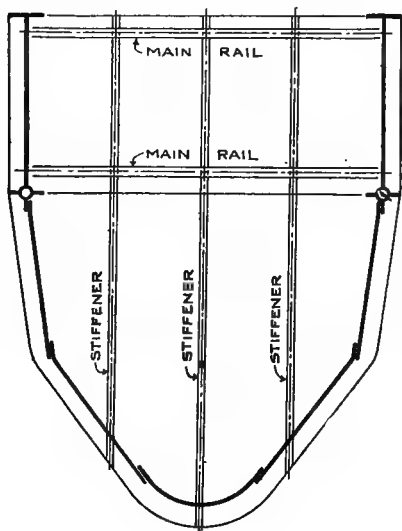


FIG. 115.

Other modifications will doubtless suggest themselves, and some may be convenient in particular circumstances. Generally, however, it should be observed that as the trough sheeting is made flatter, the effect will be either (with the sides sloping inwards) to increase the depth of the trough if the capacity is not to be reduced; or (with the sides nearly vertical) to approach the conditions of the ordinary flat-bottomed tank, necessitating a more or less extensive and costly system of framing to support the sheeting.

51. *Bulkheads*.—At the ends of the tank there must be containing walls, both to the trough and the tank proper. These end

walls are best made in the form of vertical bulkheads, after some such method as that indicated diagrammatically in Fig. 115. The vertical stiffeners may be of rolled-steel joist sections, or of bulb angles or tees, according to the circumstances of particular cases. A good arrangement is shown, in some detail, in Fig. 116; and this will need no further description. The alternative arrangement illustrated in Fig. 117 may be found convenient in some instances; and this also should be self-explanatory.

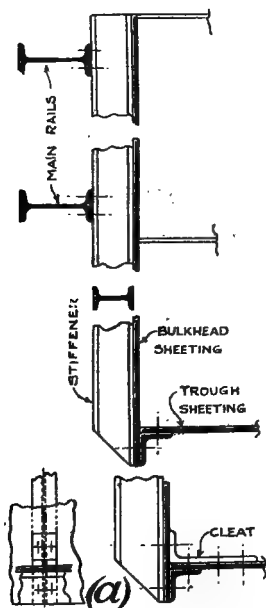


FIG. 116.

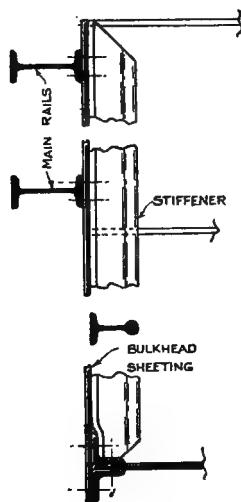


FIG. 117.

With very large troughs it may be necessary or desirable to insert an additional main horizontal rail to support the vertical stiffeners, as indicated in Fig. 118. This additional rail may press horizontally against the end stanchions, and the outward thrusts should be transmitted to the tank-wall girders by means of an inclined tie in the end panels at each side of the tank, to prevent lateral loading of the stanchions.

With fairly large troughs it may be well to provide double angles

for the connection of the bulkhead to the trough sheeting, with the bulkhead sheeting carried through to the rim of the outer angle, as shown in Fig. 119. If the vertical bulkhead stiffeners be placed on the inside of the sheeting, as in Fig. 117, the connections are rendered more difficult, and may need special treatment to ensure satisfactory results.

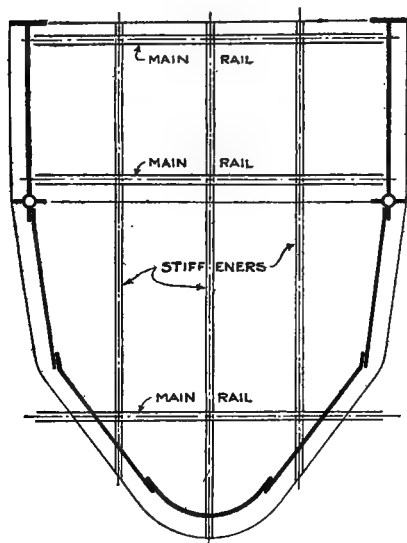


FIG. 118.

For troughs of medium or small dimensions, the single angle support, placed on the outside of the trough sheeting, as shown in Figs. 115 and 116, should provide sufficient anchorage for the lower ends of the vertical bulkhead stiffeners; though, in some cases, it may be well to provide additional cleats on the inside

of the trough, as indicated at (a) in Fig. 116.

Obviously, the angle supports at the ends of the trough must be

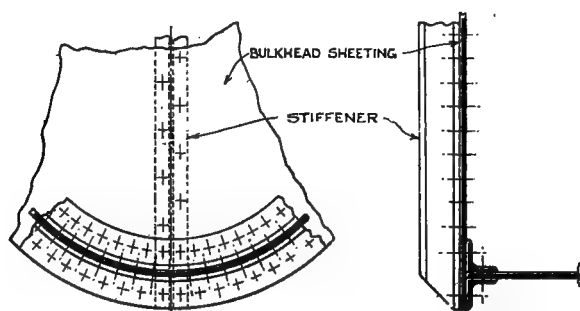


FIG. 119.

of special form, fitting around the invert plate as well as the trough sheeting. These angles should be of stout section, so that they may not bend sufficiently to permit serious opening of the seams.

The riveting in these connections, both for the bulkheads and for the trough sheeting, should be of close pitch and generous diameter throughout.

The vertical bulkhead stiffeners may be treated as propped cantilevers, and it will be clear that the arrangement of Fig. 116 possesses a real advantage (as compared with that of Fig. 117) in that the sheeting is pressed outwards against the supports, while these latter, in turn, are pressed outwards against the horizontal main rails.

52.—*Suspension of Troughs.*—The practical construction of the trough-bottomed rectangular tank presents a few problems which are worthy of consideration.

First of all comes the question of general arrangement for economy and convenience in manufacture and erection, and for conditions likely to prove favourable in working and maintenance.

Obviously, if the tank walls were placed in line with the axes of the stanchions, the latter would project into the tank as indicated in Fig. 120. This would cause difficulty in rendering the trough tight against leakage, for the trough sheeting would require to be boxed around the stanchions; and while some effective method of boxing could

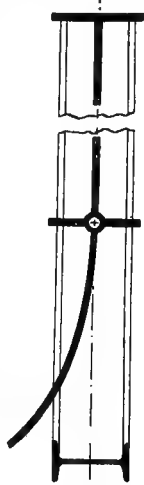


FIG. 120.

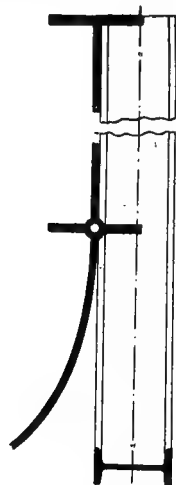


FIG. 121.

doubtless be devised, the work involved would be both troublesome and costly. Moreover, deep and awkwardly shaped pocket spaces would almost inevitably be left, in which, being practically inaccessible for either inspection or treatment, corrosive actions might go on unchecked until serious damage to the structure resulted.

To bring the stanchions closer together (transversely to the tank) would have the effect of throwing the tank walls towards the outer faces of the stanchions; and this would tend to aggravate the difficulty. Hence we are led to the remaining alternative of spreading the stanchions—by which means, clearly, the difficulty may be overcome.

The tank walls might be carried along the inner faces of the stanchions, as indicated in Fig. 121; and, provided that proper care be exercised in arranging the details, this method will probably be found more convenient and practicable than any other. An improvement may be effected by spreading the stanchions still further, sufficiently to permit the vertical limbs of the outer flange

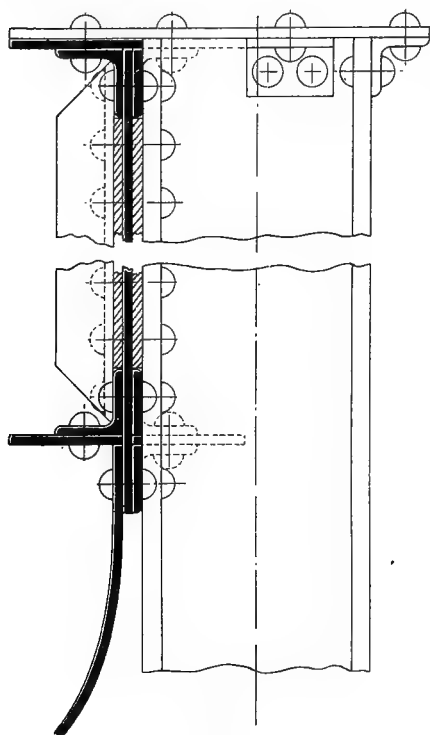


FIG. 122.

angles on the wall girders to run through, a packing strip being inserted between the stanchion flange and the wall sheeting (or webplate), as shown in Figs. 122 and 123.

The weight of the entire tank and its contents must be transmitted to the stanchions through the rivets which secure the wall sheeting to the stanchion flanges, and these rivets should therefore be designed on a generous basis. Careful attention should be given also to the preparation of the holes, as well as to the actual driving and closing of the rivets, to ensure the best working conditions.

Eccentric loading of the stanchions will be inevitable, but the transverse bracing necessary to resist the outward pressures of the con-

tained liquid may be arranged and designed to assist the stanchions; and if proper and effective use be made of this framing, and adequate anchorage provided at their bases, the stanchions should not require to be either massive or costly.

The tank walls may be designed primarily as ordinary plate girders for supporting the weight of the tank and trough, with their contents, longitudinally between the stanchions. For the upper

flange, the usual methods may be employed; but at the lower flange a slight complication is introduced by the necessity for means whereby the trough may be secured to the girder, both as regards suspension and for tightness against leakage. Obviously, under the circumstances, there are two alternative courses open. Either the webplate may project through the flange, as indicated in Fig. 122, leaving the trough sheeting to be attached to the projection by means of a simple lap joint; or the webplate may stop, as usual, and the trough be secured to the girder by means of double angle bars beneath the flange plate, as shown in Fig. 123. Of these two methods, the latter is preferable as giving more effective construction in every way, while its cost should not exceed that of the former.

Since the webplate will be subjected to considerable shearing stresses, it must be adequately stiffened against buckling; but the stiffeners between the stan-
chions may be arranged and proportioned to transmit the outward pressures of the contained liquid to the upper and lower flanges of the girder, these being designed to act as curbs between the lateral supports, bracings, or ties.

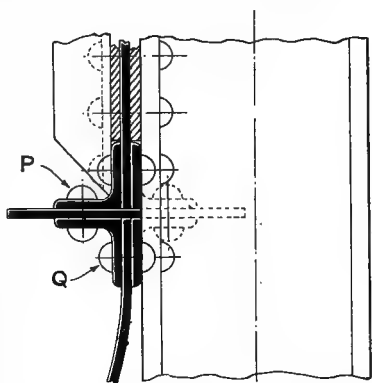


FIG. 123.

With such tanks as are likely to be suitable for this method of construction a roof will nearly always be necessary, and the framings to support the roof may be adapted to act as lateral ties to the side walls. Whatever transverse loading and bending moments be allowed to act upon the flanges between the ties must, of course, be provided for in designing; and this provision is more easily and satisfactorily made with the section of Fig. 123 than with that of Fig. 122.

For convenience in erection, with the flange arrangement of Fig. 123, the wall girder may be riveted (in the yard, before despatch to the site) as shown at (a) in Fig. 124, with a few bolts securing the inner angle and flange plate sufficiently to prevent damage in transit. The remaining angle may be either sent loose, or (preferably) bolted

to the trough—as shown at (b) in Fig. 124—for delivery. When the girder is in position, the pieces may be assembled as in Fig. 123, and the two rows of rivets P and Q driven without much difficulty.

In the longitudinal stretches between the stanchions, transverse ties to the side walls will be sufficient, since the only loading to be resisted is that due to the outward pressures of the liquid—though framing or trussing of some kind may be necessary to prevent undesirable sagging of the ties where these are of such length that they would be inconveniently large or heavy if provided with sufficient stiffness in themselves.

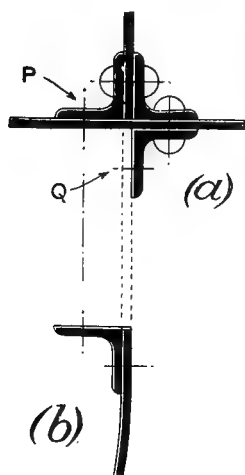


FIG. 124.

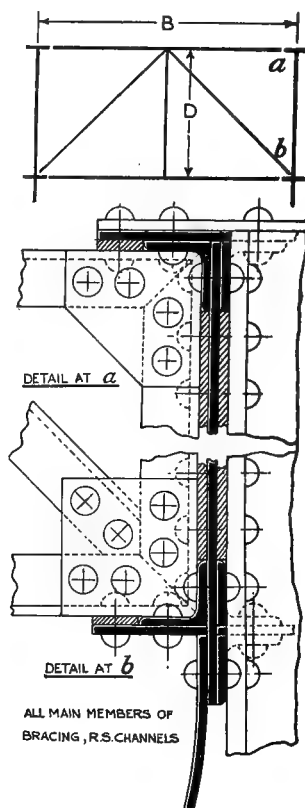


FIG. 125.

At each pair of stanchions (transversely to the tank), bracing will be necessary to assist the stanchions in taking up their loading. This bracing may be arranged as indicated in Fig. 125, the number of panels being, of course, increased where the ratio $B : D$ is more than can be properly dealt with in two panels. For obvious reasons,

the panels of the bracing should be as nearly square as may be practicable.

When designing the bracings, proper consideration should be given to all the straining actions to which they will or may be subjected, both as a whole and in their individual members. The upper boom will be placed in tension by the outward pressures of the liquid upon the tank walls, and in compression owing to the eccentric loading of the stanchions—with the arrangement similar to that indicated in Fig. 121. It is highly probable, therefore, that this member will be called upon to act as a strut under some conditions of working, and should be designed accordingly. The length as regards flexure in the vertical plane may be taken as the panel length, provided that the panel points be either themselves properly triangulated, as in Fig. 125, or adequately connected with triangulated points. As regards flexure in the horizontal plane, the length must be taken as the full width of the tank unless node points are formed by the roof framing or auxiliary bracing. For tanks not extraordinarily large, the typical details shown in Fig. 125 may be found useful; and for other cases it is probable that some simple modification of these details will meet the requirements satisfactorily. The lower boom will be placed in tension by the liquid pressures and also by the tendency to flexure of the stanchions in taking up their loading. Hence—unless the structure be subjected to some horizontal transverse loading (such as a severe wind pressure) sufficient to bring about a reversal of stress in these members—it is probable that the lower booms of the bracings will be required to act only as ties, and the suspensions indicated in Fig. 125 should be sufficient to prevent excessive sagging. The channel section is convenient for these members because it provides a simple and ready means for securing the ties to the flange plates of the wall girders; but where this is not really necessary, the ties may be of some other suitable section—such as two flat bars tacked together at intervals with bolts and distance-pieces.

At the end bulkheads, the horizontal main rails for resisting the liquid pressures may be made, by suitable arrangement, to serve also as the transverse bracings for the stanchions. Horizontal corner ties also may be used, where necessary or desirable, in the

manner described and illustrated in pages 160-163, relating to ordinary rectangular tanks.

In addition to the necessity for ensuring tightness against leakage at the connections of the end bulkheads with the tank walls and trough sheeting, the weight must be properly delivered to the end stanchions; and this calls for a little consideration.

A method which would probably meet the requirements satisfactorily in most cases is shown in Fig. 126, the sheeting of the tank and trough being continued past the end stanchions sufficiently to accommodate the circumferential angles which are to form the bulkhead connections. With this arrangement, if it be desired that the horizontal main rails of the bulkhead framing shall serve also as bracing for the stanchions, these members should be placed

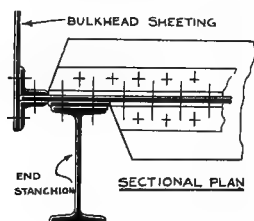


FIG. 126.

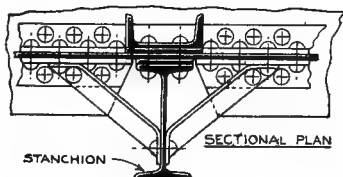


FIG. 127.

inside the tank, and in line with the end stanchions. If separate framing is to be used to form the bracing, the horizontal main rails may be placed outside the vertical supports, as indicated in Fig. 117, their loading being transmitted to the tank walls by means of suitable brackets.

To prevent wringing actions in the stanchions, angle stays may be employed, as indicated in Fig. 127, securing the outer limbs of the stanchions to the flange plates of the tank wall girders. It is desirable that such stays (or other effective support) be provided for all the stanchions, but they are particularly necessary for the end stanchions at each side of the tank, where wringing actions are likely to be set up through the action of the bulkhead framing and its connections.

The conditions as regards the stanchions will, of course, be much improved by the use of suitable bracing longitudinally, and an appreciable saving in cost may often be effected by this means

—particularly if the tank be elevated to a considerable height above the bases of the stanchions.

Other methods of arrangement and construction are possible, and will doubtless suggest themselves. For instance, the transverse framing might be made to act as a girder between each pair of stanchions, receiving the weights from the tank wall girders and transmitting them to the stanchions. Care is necessary in dealing with the intersections of these transverse girders with the tank wall girders, to ensure the proper transmission of the loading without setting up unduly high stresses in the members and their connections, and also to secure tightness against leakage; but there should be no real difficulty in the way of obtaining an efficient and economical structure on such a basis.

CHAPTER VII

CYLINDRICAL TANKS

53. *General Arrangement of Cylindrical Tanks.*—The simplest, as well as the most common, type of cylindrical tank is that which stands upon a flat base, and is used for the storage of oil, water, or other liquid, or to form the reservoir of a gasholder.

Where used for storage, such tanks are almost invariably provided with a roof, whereas in the case of a gasholder tank a roof would be both unnecessary and impracticable. Otherwise, however, there is little or no essential difference between the conditions for the design and construction of tanks for either purpose.

Tanks of this type vary from about 20 ft. to 200 ft. in diameter, and from 15 ft. to 40 ft. in height. For the small sizes (and where the use is for storage) it is open to question whether the cylindrical form is the most economical; but for the larger sizes (and for gasholder reservoirs) the cylindrical form is unquestionably preferable to any other, and it is probable that much larger tanks of this type may be expected in the future than have been constructed in the past.

The general arrangement for an ordinary cylindrical tank of moderate dimensions is indicated in Fig. 128. The cylindrical wall is constructed in strakes, the circumferential seams being always of single-riveted lap joints. The vertical seams also are sometimes made of lap joints; but it will be shown presently that economy is to be obtained by making these seams of double covered butt joints. A stout angle forms the bottom curb connecting the cylindrical wall with the flat floor plating; and a light angle curb is usually provided at the brim if there be no gallery at this level. Gasholder tanks are usually provided with a gallery, cantilevered or bracketed out from the tank, at the brim; but storage tanks

do not as a rule need such galleries unless the conditions of working are such as to prohibit the use of a roof, and at the same time to require frequent and methodical inspection of the contents.

Vertical stiffeners are generally provided, their main purpose being to prevent buckling or crumpling of the vertical sheeting; but such stiffeners may in most cases be arranged to perform other useful tasks as well—such as assisting in the support of the roof framing for a storage tank, or forming internal guides for the lowermost lift in a gasholder tank.

The floor plating, resting upon a flat and solid foundation of concrete with a sufficient covering of loose sand to prevent the weight from being supported on the rivet heads (which would leave the plates to act as a series of beams between the seams), is probably almost free from bending stresses. It is common practice to make these plates $\frac{5}{16}$ in. in thickness for all sizes—from the smallest to the largest—of tanks, and the results of experience would seem to indicate that such practice is satisfactory in ordinary circumstances of loading and use.

54. Roofs of Cylindrical Tanks.—The roof (for storage tanks) is almost invariably domed, in the form of a convex zone of a sphere, this form having been found effective in throwing off rain, as well as being suitable from other points of view. With tanks of small or moderate diameter, the roof sheeting may be supported on light trusses spanning the full diameter of the tank; and all intersecting at the centre. For larger tanks it is frequently found

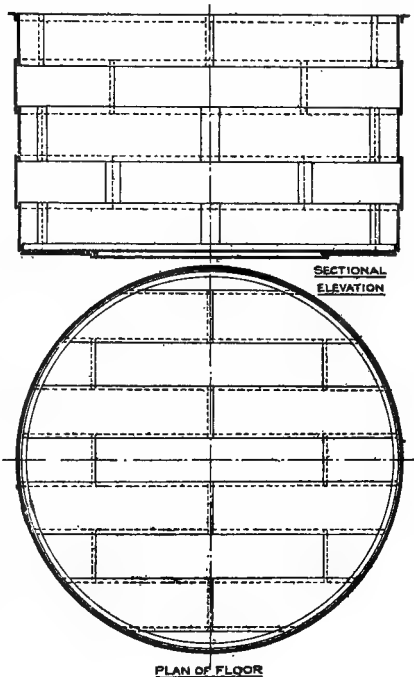


FIG. 128.

preferable to provide a central post or stanchion, with light trusses radiating from it to the circumference, as indicated in Fig. 129. Purlins are necessary, also, to provide adequate support for the roof sheeting without requiring too many trusses—which latter are, of course, somewhat costly.

With regard to the rise of the roof, there would seem to be no rule in general acceptance, some roofs being given a very sharp rise, while others appear to be almost flat. A good deal may depend, of course, upon the circumstances of particular cases; on the one hand, sufficient rise is necessary to prevent the retention of rain-water and to obtain a certain degree of strength and stiffness (even with very thin sheeting) through the “arching” action, while on the other hand it is clear that rise should be minimised to avoid waste of material and labour in the sheeting and trusses,

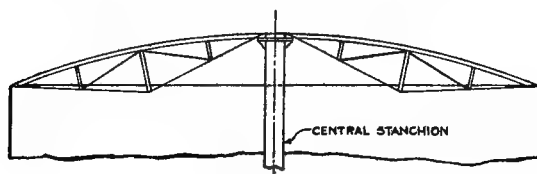


FIG. 129.

additional wind pressure, surfaces which are so steep as to be dangerous for those who have to walk over the roofs for the purpose of operating or attending to the tanks, and other undesirable effects.

For economy in preparing the roof sheeting and trusses it is obviously desirable that the spherical curving should be made to some standard radius, so that one set of forms or templets may serve for tanks of all diameters by working away from the summit in all cases. For general suitability, however, as well as for economy and efficiency in the trusses, it is clearly desirable that the rise of the roof should bear some fixed and convenient ratio to the diameter of the tank; and this would give a different spherical radius for each different tank-diameter. It is difficult to reconcile these two opposing considerations, and doubtless this difficulty is responsible for the lack of uniformity to be observed in existing tanks.

The adoption of a standard radius for the spherical roofing would result in either almost flat roofs with very shallow truss-ribs for tanks of small diameter, or unduly high-pitched roofs with excessively deep trusses for large tanks. In either case the consequence would be a need for special treatment, the cost of which would probably outweigh the saving obtained from standardising the radius of curvature.

With tanks of moderately large diameter the rise is often made about one-tenth of the tank-diameter, and this proportion is not open to any particular criticism or objection. The roof sheeting is generally very light, 14 B.W.G. being a commonly employed thickness. As a rule, the plates are not bent to their final curvature before despatch to the site; but they are, of course, shaped according to their position by tapering their side edges to radiate from the summit of the roof. At the centre, a flat circular cap-plate is used. The seams are usually single-riveted lap joints, with the lightest riveting which can be made to serve the purpose.

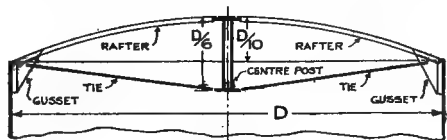


FIG. 130.

Seeing that a considerable amount of work and handling must be put upon these roof sheets, and in view of the fact that the cost of riveting is more nearly proportional to the number of rivets used than to their diameter, the author is by no means convinced that the great anxiety (which is evident on the part of tank designers) to obtain mere lightness at all costs is really productive of economy. On the basis of an all-round price per ton, of course, it is apparent that a saving in weight means a reduction in the selling price; but it is at least possible that a more just and impartial investigation of the actual costs of production might prove such a basis to be misleading in many cases—particularly in light plate work, where the weight of the material is so small that its actual cost prior to manufacture is an almost negligible fraction of that for the finished structure.

For small tanks the roof trusses usually consist of a curved rafter of light steel angle-bar and a tie-rod, without any web members other than a central strut or post, as shown in Fig. 130.

The purlins are of yet lighter angle-bars, and are put in straight between the trusses. Obviously, it would be a waste of time to attempt anything in the nature of calculation for design on the basis of loading and stresses for such construction, and it may well be thought that such methods should be considered too trumpery for use by competent and responsible engineers. At the same time, however, it is but fair to point out that failures of such roofs do not seem to occur—or, at least, to be recorded—and it is therefore difficult to substantiate any definite objection to

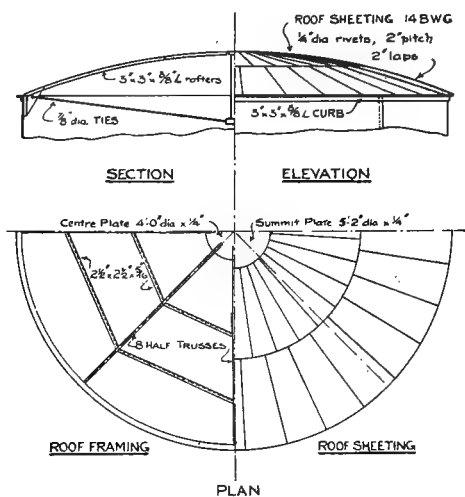


FIG. 131.

them. If (as the author believes) it could be shown, on the incontrovertible evidence of actual cost, that the adoption of other methods giving a more logical basis of stability would also reduce the costs of production, it is possible that such other methods might be used. Nothing short of an assured increase of profit is likely to bring about a change, however; and it would be unreasonable to complain against this attitude in principle, though it is sometimes carried too far.

A roof for a comparatively small tank—say, about 30 ft diameter—typical of common practice is shown in Fig. 131, and this will need no further description.

For tanks of moderate and large diameter—say, 50 ft. and over—vertical stiffeners are usually employed, to prevent buckling of the vertical sheeting. Height is, of course, a factor in the need for such stiffeners, and sometimes it may be necessary to use them in tanks of smaller diameter if the height be great; as a rule, however, the curvature of the sheeting for tanks of small diameter gives sufficient stiffness to prevent buckling under the vertical loading. The vertical loading is nearly always quite small, but its effect in setting up tendencies towards buckling in the side walls is much increased by the eccentricities caused by the circumferential seams (which are almost invariably lap joints), and the thickness of the sheeting is very small in comparison with the height of the tank.

There seems to be no generally accepted rule for these vertical stiffeners, either as to dimensions of section or spacing; but a channel section, of sufficient size to accommodate a reasonable amount of riveting, placed under the shoe of each roof truss, will be found sufficient for all ordinary cases. It would probably not be worth while to attempt anything in the nature of analytical design for such purposes.

Where the circumstances are such that access to the roof of the tank is not likely to be required except on very rare occasions, the loading is to be solely that due to ordinary atmospheric and climatic conditions, and even tightness against the ingress of water to the interior of the tank by leakage through the roof is not regarded as highly undesirable, it would be waste of effort to suggest any methods other than the very cheapest obtainable for the construction of the roof. If there are to be one or more manholes in the roof, around which men may have to work, and fairly heavy concentrated loads be applied, additional strength and stiffness may be provided locally by means of trimming or framing; and for the purposes of periodical inspection, painting, or repair, it may be assumed that adequate precautions will be taken by those responsible, to guard against possible mishap arising through weakness of the roof.

Where a somewhat higher standard is desired, or where men may frequently be required to work on many and various parts of the roof, it is possible that some form other than the spherical

might be found both suitable and advantageous. The form which most naturally suggests itself is that of the truncated cone (as shown in Fig. 132), which would seem to possess most (if not all) of the merits usually claimed for the spherical form, with the additional advantage that a standard slope of rafter may be used for tanks of all diameters, thus tending to uniformity in the processes of manufacture. A convenient rafter-slope for most cases would be

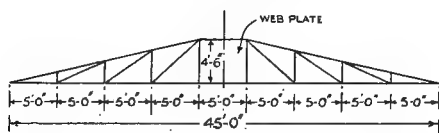


FIG. 132.

obtained by making the ratio of rise : tank diameter equal to 1 : 10, using a standard flat circular summit plate (giving a rivet circle 5 ft. diameter) for all tanks. For tanks of such diameter (say, 60 ft. and over) as to need a central stanchion to support the roof, the arrangement of trusses might be as indicated in Fig. 133.

It will be seen from Fig. 131 that a large amount of cutting and riveting is involved with the spherical doming in common use.

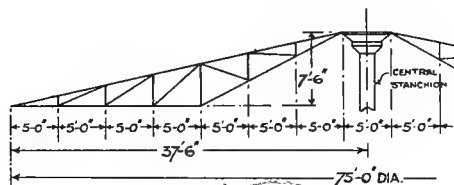


FIG. 133.

Much larger plates could be used with the conical form, thus saving a great deal of cutting and riveting, besides giving a tighter and more lasting roof.

A method of roof construction for cylindrical tanks, using the conical form, is shown in Fig. 134, and the author would suggest that the method is worthy of trial, for comparison with the methods at present employed, on the basis of economy as well as suitability and convenience.

This method would permit of standardisation to a large extent,

as will be seen, and should therefore be the means of saving time and trouble in the office, as well as in the yard and at the site. For tanks up to 60 ft. diameter, four full-diameter trusses might be used, as in Fig. 134. For all larger diameters, twelve half-trusses might be used, radiating from a central stanchion.

With four trusses, the entire loading from the roof should be supported upon two trusses at right angles, the other two trusses (in halves) transmitting their loading partly to the tank walls and partly to the main trusses at the centre. To facilitate erection,

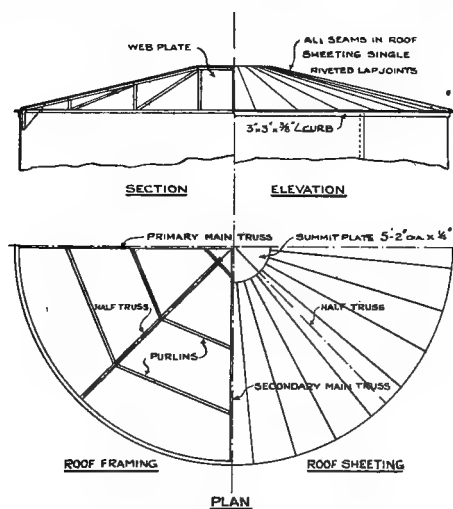


FIG. 134.

where no central stanchion is used, one main truss might be riveted up complete before lifting, provision being made in this truss for attaching the remaining trusses in halves. By means of a solid webplate in the central portion of the primary main truss, and a webplate with angle cleats on each side of the centre for the secondary main truss, as indicated in Fig. 135, the erection and fitting of the trusses could be rendered quite simple.

The longer purlins might be in the form of light trusses, as shown in Fig. 136, such an arrangement providing effective lateral support for the trusses, and thus tending to economical design.

By adopting a constant length (probably in the neighbourhood

of 5 ft.) for the panels of all trusses, working outwards from the edge of the summit plate, the purlins could be standardised for all four-truss roofs, and also for all six-truss roofs. Moreover, it is probable that by carefully selecting the panel length for the trusses, most of the purlins could be used in all roofs, whether of four or six trusses; the only necessary modification being in the struts of the trusses, which should be splayed as at (a) or (b) in Fig. 136, in order to provide easy and effective attachment for the

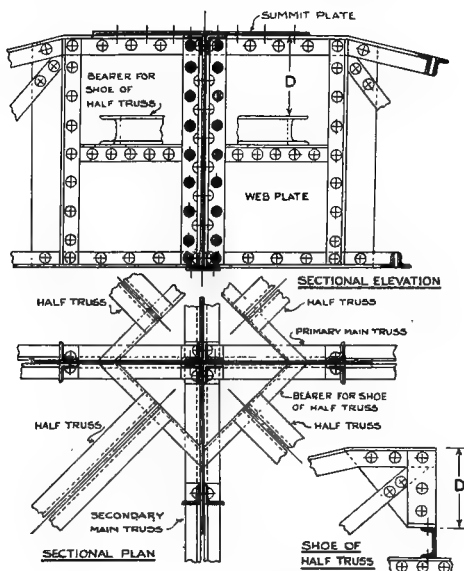


FIG. 135.

purlins. With the truss arrangement of Fig. 135, no purlin will be necessary at the edge of the summit plate; and at the next panel point the distance between the trusses will be so small that a plain angle of quite light section will suffice for the purlin. For all other panels, trussed purlins might be used, and these might well be of a fixed depth—say, about 2 ft.—for all spans. At the panel points nearest the shoes, the trusses will probably not be of sufficient depth to take a 2 ft. purlin, but a slight modification will overcome this difficulty. The trusses do not need lateral

support so close to the shoes, and hence the purlins may have the end lengths of their lower chords inclined as at (a) in Fig. 137—indeed, some such form may be preferred for all the trussed

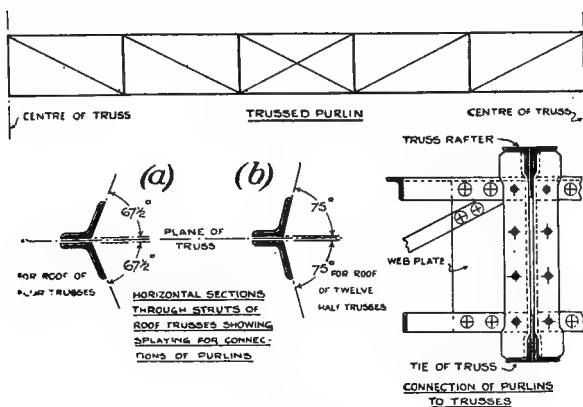


FIG. 136.

purlins, lateral support for the lower chords of the trusses being obtained (where necessary or desirable) by means of a light tie, as shown at (b) in Fig. 137.

It will be found most convenient, from all points of view, to suspend the purlins with their webs vertical; and hence, for the arrangement here suggested, the struts of the roof trusses should be vertical also.

The lengths of the purlins for standardisation may be easily calculated, and this point should not need any detailed consideration here. Their design for strength and stiffness also, with a view to standardisation, should present no great difficulty.

Since the purlins are to be straight and the roof sheeting conical, provision is necessary to ensure that the plates shall bear evenly upon the purlins for support. Such provision might well

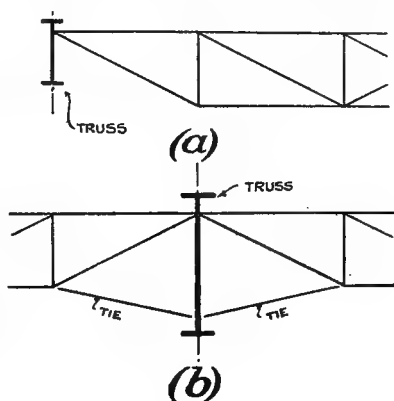


FIG. 137.

be obtained by means of hardwood packings fastened to the purlins by screws. The seams in the roof sheeting could either be arranged so that no rivets occur immediately over purlins, or rivets over purlins could be countersunk on the underside. The hardwood packings might be prepared, in strips of suitable lengths and widths, and screwed to the purlins before erection, all necessary dimensions and particulars for their preparation being obtainable by simple calculation. It might, however, be found more convenient to prepare them in the form of wedge-strips, which could be tapped into place after the sheeting has been laid in position, thus securing at once more uniform support for the sheeting and more uniform loading for the purlins. Thicker packings will be necessary for the outer sheets (by reason of the lap

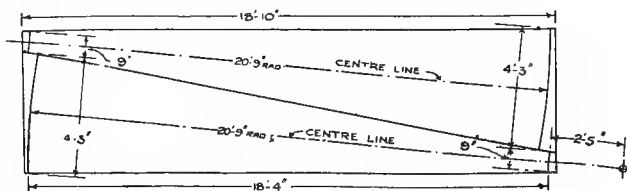


FIG. 138.

seams) than for the inner sheets, and this adjustment may be made either by means of additional strips where necessary, or full-thickness packings—the latter being preferable. This point tends to favour the wedge-strip packings tapped into position between the purlins and sheeting.

The roof sheeting might be cut from plates of suitable width as indicated in Fig. 138, thus largely reducing the amount of cutting and waste, as well as effecting a considerable saving in riveting, in comparison with the spherical doming of Fig. 131. The most convenient width of plate may be readily determined for any diameter of tank by simple calculation, and all the roof plates could then be prepared so as to be interchangeable. Thus, for a tank 40 ft. diameter, with the pitch-circle in the summit plate 5 ft. diameter, the roof sheeting to cover the top curb (of 3 in. $\times$ 3 in. angle), the calculation would be as follows—

Extreme outer circumference

$$\begin{aligned} &= \pi \times 40.5 \text{ ft.} = 3.1416 \times 486 \text{ in.} \\ &= 1527 \text{ in.} \end{aligned}$$

Taking a width of 4 ft. at the circumference of the tank as a first approximation—

$$\frac{1527 \text{ in.}}{48 \text{ in.}} = 31.8 \text{ plates.}$$

Now, there are to be eight truss-rafters; and although the roof sheeting is to bear upon the rafters, there is no need to rivet them together. Hence, it will be convenient to take the nearest even multiple of 8 for the number of plates—in this case, obviously, 32. By this means it may be ensured not only that no seam in the roof sheeting shall occur immediately over a truss, but also that each truss shall be in contact with the same kind of roof plate—*i. e.* either all inner, or all outer plates, instead of some inner and some outer—the advantages of such a course being apparent. With a larger tank, having a central stanchion and twelve half-trusses, the number should be an even multiple of 12.

Adopting 32 plates for the purposes of our example, the width at the circumference of the tank will be $\frac{1527 \text{ in.}}{32} = 47.72 \text{ in.}$ This will be the distance between the centre lines of the rivets, measured on the curve, and the necessary laps must be added on each side.

The corresponding width at the circumferential seam connecting the conical sheeting with the summit plate may now be determined. The circumference of the pitch-circle will be $\pi \times 60 \text{ in.}$, and the distance between the straight seams $\frac{3.1416 \times 60}{32} = 5.89 \text{ in.}$ A common lap for such sheeting is 2 in., and this would give a plate width about $47.72 + 5.89 + 4 = 57.61 \text{ in.}$ It would probably be found cheaper to use stock plates 5 ft. in width, giving laps just over 3 in., but saving a good deal of cutting.

A templet could be made, either in wood or steel, the dimensions being obtained by calculation from the conical form, as shown in Fig. 139, and the plates marked for cutting and holing. The

plates would doubtless need straightening and dressing after shearing—the rivet-holes might be punched before the shearing—and it would greatly facilitate the work of erection if the plates were shaped to the conical form instead of being merely flattened.

This could be done without heating the plates, by hammering in a suitable "form," and such a form could easily be made to take all plates if a standard slope for the conical roof were adopted, distances from the summit plate being marked in the trough to indicate the position in which any particular sheet is to be laid to suit a given tank diameter.

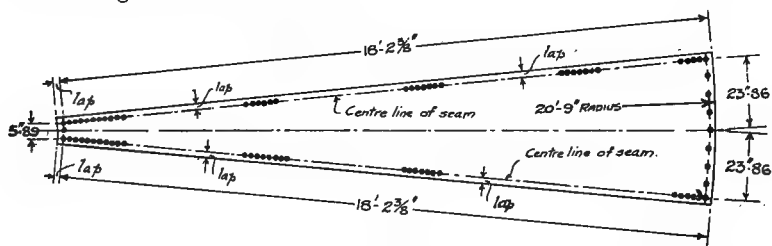


FIG. 139.

It would almost certainly be found cheaper to use packings between the summit plate and the inner conical plates, instead of thinning out the corners of the outer plates to allow for the lap-joints; and similarly, to pack between the top curb and the outer conical plates at the circumference of the tank.

For marking the plates, it is probable that the employment of one sheet, carefully set out as a "master," would be found preferable to the use of a wooden templet. Methods for simplifying the work of setting-out—such as the preparation of a set of stock "curves," varying in radius by 6 in. or 1 ft., with centre lines clearly marked, as indicated in Fig. 140—will doubtless suggest themselves.

The length of each plate might be easily calculated from a diagram, such as that of Fig. 141, showing the "rise and going" dimensions of the conical slope. For the 40 ft. tank considered above—

Length of plate

$$\begin{aligned}
 &= \sqrt{(17.75)^2 + (4.0)^2} = \sqrt{315.0625 + 16} \\
 &= \sqrt{331.0625} = 18.2 \text{ ft.} = 18 \text{ ft. } 2\frac{3}{8} \text{ in.;}
 \end{aligned}$$

adding $7\frac{5}{8}$ in. for lap at the summit plate and to allow for cutting to curve at the outer circumference, a suitable and convenient length would be 18 ft. 10 in., as shown in Fig. 138.

The riveting for tank roofs varies considerably with different manufacturers. With 14 B.W.G. sheeting, $\frac{1}{4}$ in. rivets at 2 in. pitch are common; but with thicker plates, and if the roof is to be no more than a lid, both diameter and pitch may be increased with advantage.

Obviously, much will depend upon the circumstances of each individual case in determining the necessary riveting. If the liquid to be stored is water, no very elaborate precautions against either the ingress of moisture or evaporation of the contents may be necessary. On the other hand, if the tank is to contain some highly volatile liquid, or some substance which would be damaged by the leakage of rain or snow, care must be taken to render the roof secure against such faults. These are special matters, however, and therefore need not be discussed at length here.

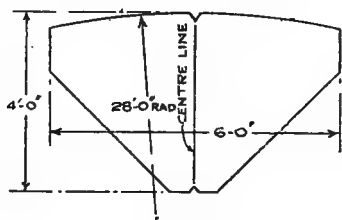


FIG. 140.

55. Floors of Cylindrical Tanks.

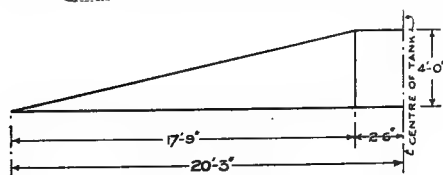


FIG. 141.

As already stated, the floor plates in tanks which stand upon a flat bed of concrete are usually $\frac{1}{4}$ in. or $\frac{5}{16}$ in. in thickness for all diameters of tank. The seams are of single-riveted lap joints, with 2 in. or $2\frac{1}{2}$ in. laps; and the corner of the

middle plate is thinned out (as described and illustrated in Chapter III, Fig. 31) to provide for the junction of three plates. For the rivets, $\frac{5}{8}$ in. diameter at 2 in. pitch is fairly representative of common practice, though departures may be found in some cases.

Clearly, if the plates were uniformly supported throughout the floor, the main function of the riveting would be to ensure tightness against leakage under the full liquid head. The only straining

actions to which the rivets would be subjected in such circumstances would be those due to such causes as initial and secondary stresses, and the effects of temperature changes. As will be shown presently, however, uniform support for the floor is not likely to be realised in commercial tanks; and hence the rivets are in all probability subjected to more or less severe straining actions through the bending of the plates. But if reasonable care be taken in erecting the tank, the plates cannot bend much without finding improved support; and this factor, combined with the necessity for closely pitched rivets to ensure tightness, renders

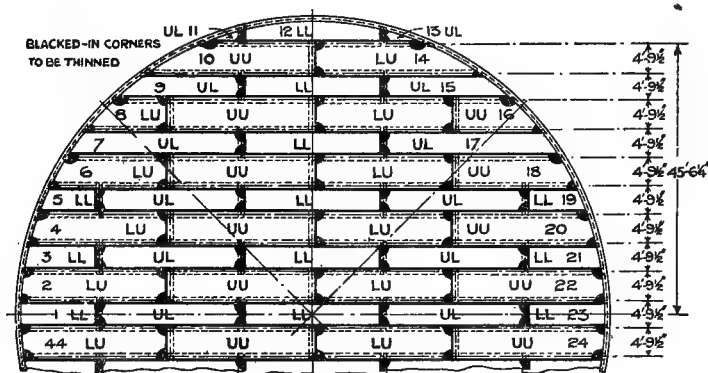


FIG. 142.

treatment of the seams for strength unnecessary in all but exceptional circumstances.

The plates, obviously, should be as large as possible consistent with economy and convenience in manufacture, handling, and transport, for by this means riveting may be minimised. Dimensions in common use are 24 ft. by 5 ft., these having been found most suitable from all points of view. The layout should be such as will give all (or as nearly as possible all) the rectangular plates of uniform shape and size, while reducing to a minimum the unavoidable cutting, waste, and riveting, for the shaped plates around the circumference.

A plan for a small floor is indicated in Fig. 128, and one for a medium-sized floor—100 ft. in diameter—in Fig. 142.

In laying out the floor-plates, the number of rows may be odd (as in Fig. 142) or even (as in Fig. 143), according to the ratio borne by the diameter of the tank to the effective width—*i. e.* the distance between the centre lines of the long seams—of the floor-plates.

Taking the tank as 100 ft. diameter over the bottom curb, for instance, with the centre lines of the seams 4 ft. 9½ in. apart, the number of plates would be $100 \div 4\frac{19}{24} = 2400 \div 115 = 20.82$. A suitable arrangement, then, might be obtained with 19 full-width rows (giving a total width of $19 \times 4\text{ ft. } 9\frac{1}{2}\text{ in.} = 91\text{ ft. } 0\frac{1}{2}\text{ in.}$),

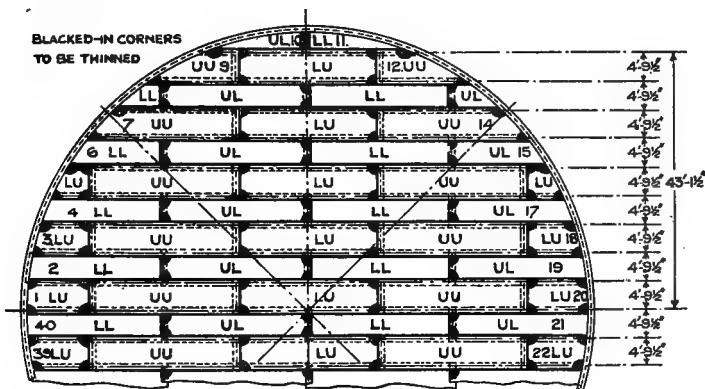


FIG. 143.

leaving the two shaped plates to make up the remaining 8 ft. 11½ in. —or, rather, say, 8 ft. 11 in., since the plate should not extend to the face of the curb—between them.

With a tank 95 ft. diameter, the arrangement might be as shown in Fig. 143, using 18 full-width rows; or, alternatively, 19 full-width rows might be used, with smaller (extreme) shaped plates, as in Fig. 144. The total length of seams with the former arrangement is approximately 1632 ft. (1364 ft. in the long seams, and 268 ft. in the 5 ft. seams); while the latter plan has about 1729 ft. run of seams (1468 ft. in the long seams, and 260 ft. in the 5 ft. seams). Hence, it would appear that a saving of nearly 6 per cent. in the riveting may be obtained by adopting the arrangement of Fig. 143, instead of that shown in Fig. 144, for such a case.

There are, however, other factors needing consideration in determining the most suitable layout for a tank floor; and although 6 per cent. saving in the total length of seams represents an appreciable item in a large floor, the advantage may easily be lost if the layout involve a need for special care through some departure from the usual course of handling or erection.

One such factor is the influence exercised, both in manufacture and erection, by a basic symmetry of arrangement, and uniform order of procedure. If some tank floors have an even and some an odd number of rows, confusion may arise through the former having a seam along a diameter of the floor, while the latter have

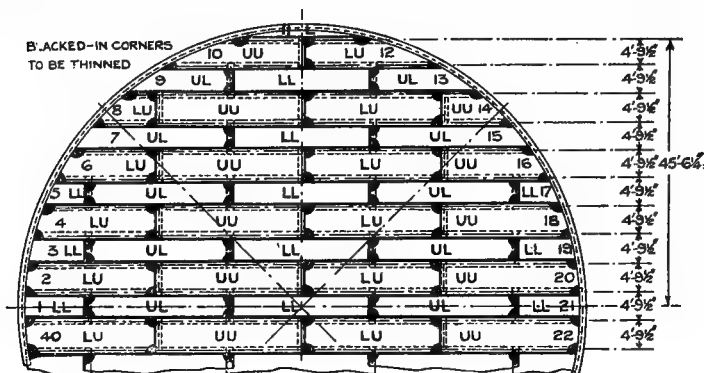


FIG. 144.

not. With a layout such as that indicated in Figs. 142 and 144, a pivotal plate may be laid down at the centre of the floor, and the work of assembling and riveting may then proceed symmetrically along lines radiating outwards in all directions, with obvious advantage; whereas with the arrangement of Fig. 143 there is no such symmetry. Further, in the floors of Figs. 142 and 144 the shaped plates around the circumference will be exactly alike in fours as regards shape, dimensions, and setting out for riveting; whereas with the arrangement of Fig. 143 these plates will be alike in pairs only as regards shape, dimensions, and rivet centre lines.

This means a relatively large increase in trouble throughout the office work, manufacture, transport (including such incidentals

as marking, sequence of delivery, schedules, marking plans, etc.), and erection, the cost of which must obviously be considerable.

It is probable that the best results from all points of view are likely to be secured by the adoption of a layout similar to those indicated in Figs. 142 and 144 for tanks of all diameters, unless special and exceptional circumstances render a departure necessary or desirable.

The plates marked 7 and 17 in Fig. 142 would be more than 24 ft. in (extreme) length; but in most cases it will be found preferable to have them thus longer than the rectangular plates—provided they be not so long as to involve disproportionate extra cost for rolling or transport—rather than to introduce additional transverse seams by using shorter plates. This is a point which arises frequently in arranging the layouts for such floors, and it often needs careful treatment to ensure the most satisfactory results.

In Figs. 142, 143, and 144, the plate corners requiring to be thinned are indicated by blacked-in quadrants. As will be seen, the rows are laid alternately “lower” and “upper”—thus, the row 1—23 in Fig. 142 is a lower row throughout; the row 2—22 an upper row throughout; row 3—21 lower; row 4—20 upper; and so on. Within the rows themselves the plates will be alternately upper and lower if lap joints be used throughout. Hence, the pivotal plate at the centre of the tank will be a *lower plate in a lower row* (indicated by the mark “LL”); while those at each end of it will be *upper plates in a lower row* (indicated by the mark “UL”). In the next row (2—22) the order will be alternately a *lower plate in an upper row* (marked “LU”), and an *upper plate in an upper row* (marked “UU”).

Now, it will be clear that the upper plates in a lower row (UL) must have their corners thinned to allow the upper-row plates to lie flat along those of the lower rows, while the lowermost plates (LL) must have their corners set downwards to accommodate the thinned corners above them. Similarly, the lower plates in an upper row (LU) must have their corners thinned, while the uppermost plates (UU) must have their corners set upwards to accommodate the thinned corners beneath them.

It should be noticed that the object in view is to keep the

upper surface of the lower rows, and also the lower surface of the upper rows, flat throughout the floor. On this basis the thinned corners and the set corners may be located easily. As will appear from an inspection of Fig. 142, every plate corner in the floor is either a thinned corner or a set corner.

Around the circumference of the floor, the plates meeting the curb will be alternately upper-row and lower-row plates. The lower surface of the curb angle should be regarded as lying at the same level as the upper surface of the lower-row plates which are to be connected with it, and the upper-row plates must have their curb corners thinned to make their way between the lower-row plates (which must have their corners set downwards) and the curb.

Thinning and setting the corners of such plates are inevitably an expensive proceeding. A plate 24 ft. by 5 ft. is an awkward and troublesome thing to hold for the heating and working of its corners one (or even two) at a time. Corners must be heated for thinning; and although the set corners are commonly ignored in the yard (the plates being sent to the site flat, leaving the sets to be imposed by the riveting), it is probable that the cost of the job might be reduced rather than increased if these corners were properly set in the yard before despatching the plates to the site.

Considerable expense is incurred also on almost every job through time spent on these thinned-corner junctions, both in coaxing the plates into position for riveting, and also afterwards in persuading them to abstain from leaking.

Wedge-shaped packings (see Fig. 32) might be used instead of the thinned corners; but it is open to doubt as to whether the cost of a floor would be reduced by such means. The packings themselves are somewhat costly to make, while they render the set corners more complicated than they are with the thinned corners. Moreover, wedge-shaped packings do not by any means simplify the work of assembling the floor-plates; and they sometimes give rise to leaks which are very troublesome to cure.

In passing, it is worthy of notice that, with the corners of the lowermost plates all set downwards, the plates alternately "upper" and "lower," and the rivet heads projecting at different levels, the underside of such a floor is by no means flat; and hence, the

common assumption that the floor-plates will be free from bending actions is not likely to be realised. Even with a fairly thick layer of sand spread evenly over the concrete base, the loading cannot be distributed with any degree of uniformity; and bending actions in the plates are therefore inevitable with a layout such as those indicated in Figs. 142, 143, and 144.

The author recommends, as tending towards facility and economy in the manufacture and erection of such floors, that the transverse (5 ft.) seams be of single-riveted butt joints, with inside covers,

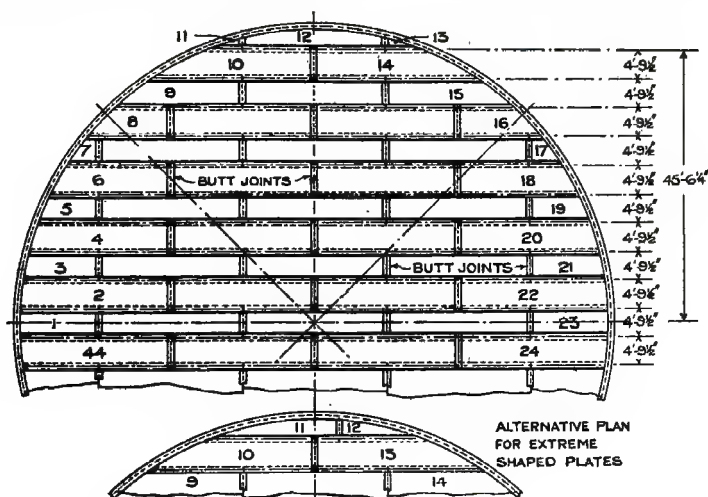


FIG. 145.

instead of lap joints. By this means all thinning and setting of plate corners would be obviated, packing strips being inserted between the bottom curb and the lower floor-plates to fill the spaces caused by the use of lap joints for the long seams. A layout on this basis for a 100 ft.-diameter tank is shown in Fig. 145. The butt covers for the transverse seams would not involve any more material than lap joints; and the additional cost for the extra holing and riveting should not represent more than a low percentage increase, since in the layout of Fig. 144 the total length of the transverse seams is but 260 ft. (for the whole floor), as against 1470 ft. for the long seams. Adding to this considera-

tion the fact that each butt cover could be riveted to one plate in the yard, it will be seen that the proposal should give an appreciable saving in cost, as well as a distinctly better job. Each pair of adjacent rows would come together without need for coaxing to form the long lap joints, and thus a good deal of time should be saved in erection.

The butt covers (and also the packing strips at the curb) would need a little care in preparation to ensure their fitting the lap joints properly—without requiring to be chipped, and without leaving gaps during assembly at the site—but this need not cost much if taken in hand with intelligence. Using inside covers only, those for the upper-row plates would extend the full width of the

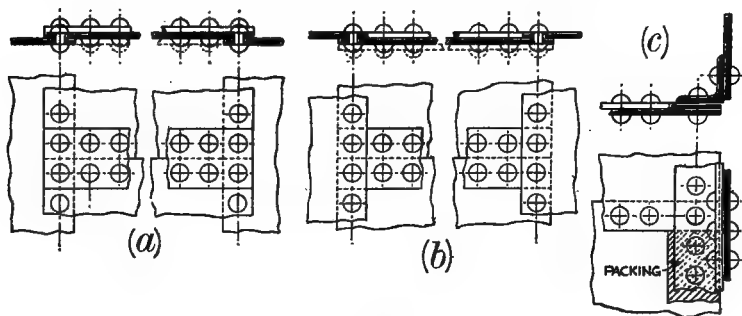


FIG. 146.

plates, taking two of the lap-seam rivets at each end, as shown at (a) in Fig. 146; while those for the lower-row plates would fit between the lap joints at each end, as at (b) in Fig. 146. Alternatively, the butt covers for the lower-row plates might be fitted on the underside, and extend the full width of the plates, becoming thus exactly like those for the upper-row plates. The main objection to this latter method is that the plates might not butt together truly, and leakage into the joint—if not through it—might be difficult to prevent in consequence. However, the inside cover also is open to a similar objection in that leakage at its ends could find an easy escape owing to the butt not being covered on the outside. Beyond question, the best method would be to use double covers for the butt joints of all lower-row plates, as shown

dotted at (b) in Fig. 145; and even with this refinement it is probable that the floor would be actually cheaper than with the plate corners thinned and set for the use of lap joints throughout.

The packings between the curb and the lower-row plates might be arranged as indicated at (c) in Fig. 146, and if reasonable care be taken in preparing them it should be a comparatively simple matter to secure tightness by means of caulking. These packings could, doubtless, be all formed of the "waste" from the shaped plates around the circumference. A floor constructed on this basis would obviously find more uniform support under ordinary conditions than could one of the type illustrated in Figs. 142, 143, and 144.

A point of practical interest and importance arises in connection with the shaped plates around the circumference of the floor.

For ordering these plates from the mills, a method commonly employed is to set down the layout for the complete floor—or so much of it as is repeated—to a fairly large scale; sometimes it is even laid out full size on the templet floor. The necessary length of each plate, to allow for shaping, is measured from the layout, and the plates are then ordered rectangular, with the result that not only is a good deal of waste material purchased, but a more or less considerable sum is paid for the carriage and handling of that waste, while the plates have still to be shaped after delivery.

Now the plates may be at least as cheaply shaped at the mills if the necessary particulars be supplied; and although the waste material must be paid for (which is but fair), transport costs will be reduced, while an appreciable saving both of time and trouble should be effected by having the plates correctly shaped, and marked for position in the floor, before delivery.

The necessary particulars may be readily obtained by means of simple calculation; and with a little care and thought they may be supplied to the mills in such a manner as to ensure satisfactory results.

Working on the main rivet centre lines as chords intersecting a diameter of the circle at right angles, the lengths of these chords may be calculated from the known dimensions; for the case is then

similar to that illustrated in Fig. 147, where, since $c^2 = V(D - V)$,

$$c = \sqrt{V(D - V)}.$$

The value of D is, of course, constant for all the chords in a given tank; and the appropriate value of V may be determined easily for each chord.

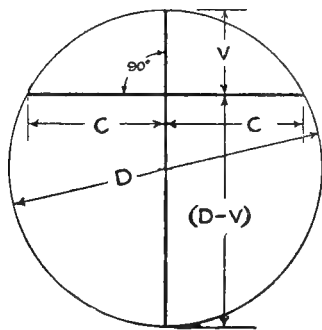


FIG. 147.

Consider, for example, the plate marked "8 L U" in Fig. 142. Taking the tank as 100 ft. diameter over the bottom curb, and allowing for the edge of the floor plating to be set in $\frac{1}{4}$ in. behind the face of the curb, D for our purpose will be 99 ft. 11 $\frac{1}{2}$ in.

Treating the upper chord first, V will be {49 ft. 11 $\frac{3}{4}$ in. - 7 $\frac{1}{2}$ (4 ft. 9 $\frac{1}{2}$ in.)} = 49 ft. 11 $\frac{3}{4}$ in. - 35 ft. 11 $\frac{1}{4}$ in. = 14 ft. 0 $\frac{1}{2}$ in., while $(D - V)$ will be 99 ft. 11 $\frac{1}{2}$ in. - 14 ft. 0 $\frac{1}{2}$ in. = 85 ft. 11 in. Then—

$$\begin{aligned} c &= \sqrt{14\frac{1}{2} \times 85\frac{1}{2}} = \sqrt{\frac{337}{24} \times \frac{1031}{12}} \\ &= \sqrt{\frac{337}{24} \times \frac{2062}{24}} \\ &= \frac{\sqrt{337 \times 2062}}{24} = \frac{\sqrt{694,894}}{24} \times \frac{833.60}{24} \\ &= 34.73 \text{ ft.} \\ &= 34 \text{ ft. } 8\frac{3}{4} \text{ in.} \end{aligned}$$

It is not much use working to greater refinement than the nearest eighth-of-an-inch in such matters; for not only will this give as good a job as is necessary for the purpose, but the use of sixteenths tends to confuse rather than to simplify the marking, without giving any greater accuracy in shearing. The approximation should, of course, be reserved until the end of the calculation.

Various methods for performing the arithmetical work will doubtless suggest themselves, and some may be more convenient than others in particular cases. That shown in the above calculation is preferred by the author for ordinary use; it is simple,

and possesses other good points which will be apparent on consideration.

Assuming that the rectangular plates are of such length that the centre lines of the short seams are 24 ft. apart, the length of the plate under consideration will be 34 ft. $8\frac{3}{4}$ in. — 24 ft. = 10 ft. $8\frac{3}{4}$ in. at its upper edge.

For the lower chord, V will be 14 ft. $0\frac{1}{2}$ in. + 4 ft. $9\frac{1}{2}$ in. = 18 ft. 10 in., and (D — V) will be 85 ft. 11 in. — 4 ft. $9\frac{1}{2}$ in. = 81 ft. $1\frac{1}{2}$ in. Hence—

$$\begin{aligned} c &= \sqrt{18\frac{10}{12} \times 81\frac{3}{4}} = \sqrt{\frac{226}{12} \times \frac{1947}{24}} \\ &= \sqrt{\frac{452}{24} \times \frac{1947}{24}} \\ &= \frac{\sqrt{452 \times 1947}}{24} = \frac{\sqrt{880,044}}{24} = \frac{938.11}{24} \\ &= 39.09 \text{ ft.} \\ &= 39 \text{ ft. } 1\frac{1}{8} \text{ in.} \end{aligned}$$

Subtracting the length of the rectangular plate, the required length of the plate under consideration, at its lower edge, will be 39 ft. $1\frac{1}{8}$ in. — 24 ft. = 15 ft. $1\frac{1}{8}$ in.

In making the calculations for a floor, it is well to work on a definite plan and sequence, tabulating all important data as obtained. The work may be expedited by such means, while errors are less likely to occur, and checking is facilitated.

A schedule of the shaped plates should be supplied to the mills, giving a dimensioned sketch for each plate with its marking. Thus, the sketch for the plate "8 L U" of Fig. 142 would be as shown in Fig. 148, and the schedule would indicate the number and particulars of the plates to be cut to the given dimensions. For instance, with the floor of Fig. 142, there would be another plate, marked "30 L U," exactly similar in all other respects to that shown; and two others to the same dimensions but "opposite hand," one marked "16 U U" and the other marked "38 U U." It would not be necessary to mark each of these plates separately from the sketch; one could be marked and sheared, and this would serve as a templet for the other three plates. The "opposite

hand" effect may be obtained by marking off these plates on the reverse side—and even this refinement may be dispensed with if there be no objection to the reversal of the effects due to shearing.

The markings "L U" and "U U" quoted in the foregoing description relate, of course, to the thinned-corner plan of Fig. 142. With the short seams butt jointed, as in Fig. 145, the row-marks need only be "L" (indicating "lower row") and "U" (indicating "upper row") in addition to the positional numbers.

A templet-curve, of 49 ft. $11\frac{3}{4}$ in. radius for the case considered above, should be supplied with the schedule, and clear instructions

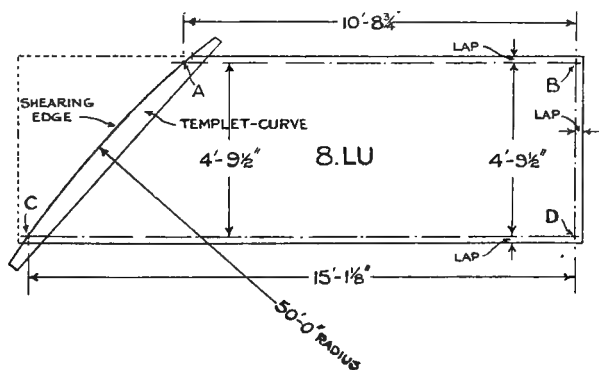


FIG. 148.

given for its use in marking-off. Using Fig. 148 for reference, these instructions might be as follows: "Set out the main rivet centre lines A B and C D, 4 ft. $9\frac{1}{2}$ in. apart, truly parallel with each other and as nearly so with the plate edges as may be. Set out the transverse rivet centre line B D, truly perpendicular to A B and C D, and providing $1\frac{1}{4}$ in. (if lap joints are to be used for the short seams) for lap. Carefully lay off the dimensions A B and C D shown in the sketches, and apply the templet-curve provided (give distinguishing number or other mark on templet-curve, and clearly indicate the marking edge) to obtain the curved line A C to which the plate is to be sheared."

This is a typical case in which considerable economies may be effected through the exercise of proper care and skill in ordering material.

If the short seams are to be butt joints, as indicated in Fig. 145, modification in the stated lengths will be necessary, it being obviously desirable that the lines used for setting out shall be rivet centre lines rather than plate edges. Such modification is, however, too simple to call for detailed description here.

The templet-curve (which should be made in duplicate, one being retained as proof in the event of dispute or error) should be carefully protected against damage in transit. It may be made of thin sheet steel, and if laid between two stiff boards—one suitably recessed to receive the templet—should come to no harm in ordinary circumstances. If the templet-curve be cut from a straight parallel strip of sheet, and the back edge left straight, as shown in Fig. 149, there can be no confusion or doubt as to which is the marking edge.

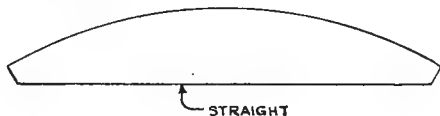


FIG. 149.

A stock set of templet-curves could be made, so as to be ready for use when required, and it is probable that no great harm would result from using the nearest 6-in. radius for floors of moderate and large diameters.

The templet-curve may be set out by that simple method—which, by the way, appears to be nothing like so well known nowadays as it was to the wily and resourceful (if somewhat secretive) templet makers of a few years ago—based on the fact that the angle in a circular segment is constant. With the addition of a further device from Euclid—which device the author believes is by no means so well known as it deserves to be—this method may be made to yield thoroughly satisfactory results without the need for large and costly floors on which curves may be struck.

This device consists in an application of the fact that the angle in a segment is equal to the alternate angle between the chord and the tangent to the circle at its extremity—thus, in Fig. 150, the angle ABC is equal to the angle DAC . This fact may be utilised in constructing the profile board which is to represent the angle

in the segment, and which (provided with a suitable scribe fixed at its apex, and made to swing into all positions possible for the segment angle by means of a nail driven at each end of the chord A C) will strike out the circular arc A B C to the required radius.

Suppose the curve is 50 ft. radius, and the templet is to be about 8 ft. (a convenient size) in length. Then, take (Fig. 150) $A E = E C = 5$ ft.—*i. e.* 1 ft. more than half the length of the templet—and set this down on a flat board, as indicated in Fig. 151, clipping the templet strip in position as shown.

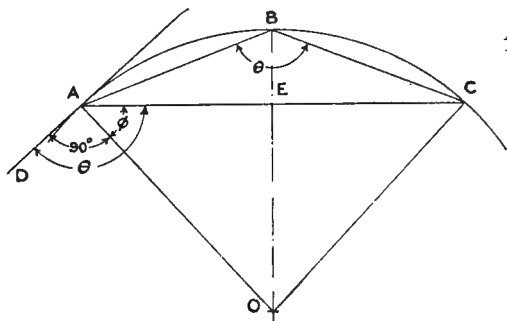


FIG. 150.

Now (Fig. 150), $\frac{A E}{A O} = \cos \phi$, and since both A E and A O are known, $\cos \phi$ may be calculated. For the case proposed, $\cos \phi = \frac{5}{50} = 0.1$; whence, from the tables, $\phi = 84^{\circ} - 15'$. Adding to this 90° (*i. e.* the angle D A O), $\theta = 84^{\circ} - 15' + 90^{\circ} = 174^{\circ} - 15'$; and this is the magnitude of the angle A B C in the segment. Subtracting this from 180° , the remainder is $5^{\circ} - 45'$, and halving this remainder, $2^{\circ} - 52'$ is obtained for each of the angles B A C and B C A. From the tables, $\tan 2^{\circ} - 52' = 0.0501$; and since A E = 60 in., B E = 60 in. $\times 0.0501 = 3.006$ in.

On a second piece of board, as indicated at (a) in Fig. 151, set out the isosceles triangle A B C to these dimensions, making this board about 20 ft. in length. Cut this board to the triangular shape set out, but leave a substantial piece on around the apex B, and fit a scribe or pencil through the board at B. If nails be driven at A and C on the first (or base) board, and the second board

(cut to the triangular shape) be made to slide round so that its two limbs are always in contact with the nails, the scribe or pencil at B will sweep out the circular arc required on the steel strip for the templet.

Instead of a solid board, as at (a) in Fig. 151, a light frame may be built up of battens to the required dimensions; or a shorter chord may be taken—say, 2 ft. 6 in. instead of 5 ft.—and the curve drawn in two parts.

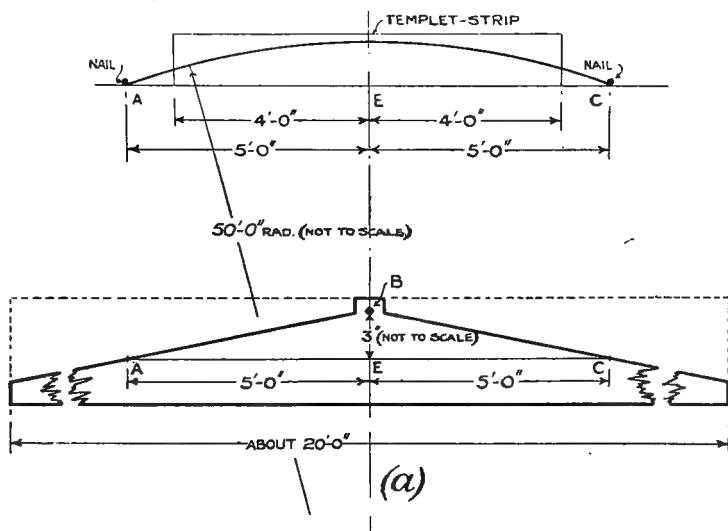


FIG. 151.

Another method which might be used is by the calculation of offsets on a chord.

To avoid the use of a very long templet-curve, intermediate chords may be calculated for the plates at the ends of such rows as 9—15, and 10—14, in Fig. 142; while the extreme plates—such as those marked 11, 12, and 13 in Fig. 142—may be given two or more intermediate chords as necessary.

If the rivet centre-lines be laid out symmetrically about two mutually perpendicular diameters of the floor, the markings will repeat on plates similarly placed. Hence, the plates may be regarded as forming groups, and one plate may be marked and

drilled to serve as a master for all others of the same group. Moreover, a large proportion of the marking for the main rectangular plates forming the body of the floor should repeat on all plates; so that much of the drilling may be done from a single master plate.

For the shaped plates, around the circumference, the holes for the straight riveting should be drilled—or at least set out—from a rectangular master plate so far as possible, any specially spaced holes to fill in broken pitches near the curb riveting being left for

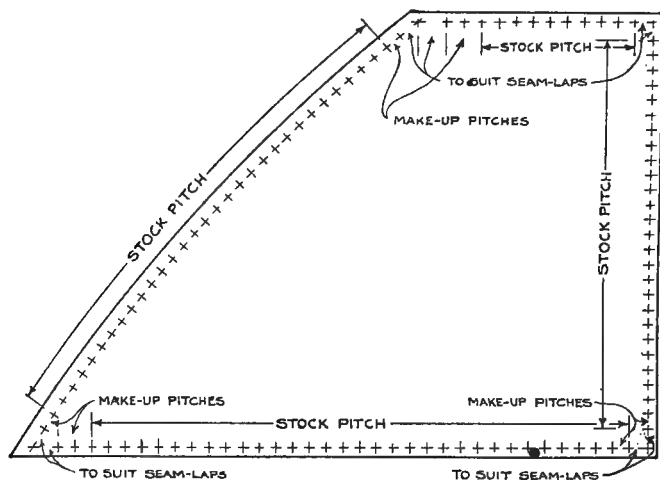


FIG. 152.

drilling (or punching) separately. The curb riveting should be set out with extreme care, and drilled with precision, on each plate which can be used as a master for others. A "left-hand" plate may be drilled from a "right-hand" master by turning back-to-front, provided there be no special features which would be rendered inaccurate by such treatment.

A rivet should occur at each intersection of the straight and curved pitch-lines, as indicated in Fig. 152; the intermediate rivets being set out to suit the long seams, and at the same time to give as nearly uniform pitch as possible.

By this means the setting out and drilling may be performed

without requiring large floor area; and with such accuracy as will ensure the plates coming together easily, and true to dimensions, when assembled at the site.

The bottom curb angle, should, if possible, be drilled with the plates to which it will be riveted, for obvious reasons; and it should be curved to its proper radius before drilling. Internal covers, of sufficient length to prevent leakage, should be provided to all joints—which may be plain butts—in the curb angle. Clearly, the curb should be in the longest lengths consistent with facility and economy in the handling and transport.

56. Erection of Floors on Solid Bases.—“Erection” is, perhaps, not a good word for the processes involved in the final preparation and fixing in position of the floor for a tank to stand directly upon a base of concrete at ground level; but it is sometimes less dangerous to use a poor word which is in common acceptance than to introduce a new one.

The riveting of the floor-plates to each other and to the bottom curb involves a practical difficulty in the need for sufficient clearance beneath the plates to permit the insertion and holding-up of the rivets—indeed, the rivet furnaces themselves must be accommodated beneath the floor if good riveting is to be obtained.

This usually means that, with a floor of moderate or large diameter, the plates must be supported at some considerable height (not less than 4 ft.) above the concrete base until the riveting of the floor is complete; and afterwards the whole floor must be lowered into position.

One commonly employed method for effecting this is to form circular holes, about 1 ft. diameter, in the plates, through which vertical screws may project, each engaging with a bridge-piece spanning the hole, as indicated in Fig. 153. After the floor has been lowered into position upon its base, the screws and bridge-

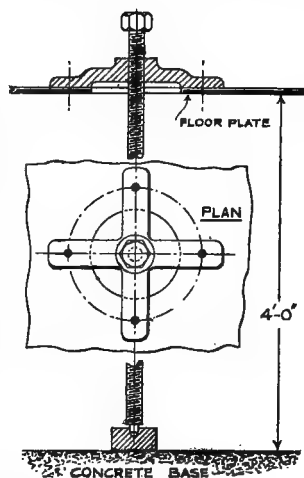


FIG. 153.

pieces may be removed, and the circular holes closed by means of cover-plates bolted to the floor-plates.

It will be obvious that the plates are inevitably subjected to uneven straining actions while supported in this manner, and even more so during lowering. Moreover, there can be no certainty that the floor will find anything approaching uniform bearing upon the concrete base when lowered; and while it is but fair to state that trouble seldom seems to arise from this cause, it is not possible to regard the method as satisfactory. Perhaps the most serious charge which lies against this method is the large amount of time and labour occupied in lifting and supporting the plates, and lowering the whole floor. The cost of this incidental work is, obviously, considerable.

Sometimes, where plenty of space is available, the floor is riveted while supported on sleepers at the side of the concrete base, and rolled into position when complete. The gear necessary for the rolling, and the straining actions applied to the floor, render this method even less worthy of approbation than that described above; while it is seldom that a site affords the necessary space.

With small floors the plates may be suspended from the lifting tackle to be used afterwards for hoisting the wall-plates and roof; but even then it is inevitable that the floor is subjected to highly undesirable straining actions while being lowered into position.

The whole problem calls for consideration with a view to finding improved methods; and it is to be hoped that those who are interested will soon make some real effort to obtain a practical solution. The author is inclined to the view that there is in this direction a promising opportunity for the employment of electrical or flame welding; and if some simple arrangement were devised, and the cost of the welding made reasonably comparable with that of ordinary riveting, the saving in time and trouble should offer a sufficient inducement for tank constructors to try the experiment. It goes without saying that there are difficulties to be overcome, and the plates would doubtless need special preparation; but, on the other hand, the cost of holing and riveting would be saved, and if the resulting floor were satisfactory as regards durability and tightness against leakage, even some additional outlay in respect of the actual work might be more than repaid by the saving in

time effected. Possibly some such arrangement as that indicated in Fig. 154 might form a basis for trial, the plates fitting against function strips of T-section at all seams. Small folding wedges might be used at intervals, as shown, to act as cotters in holding the plates and strips firmly in position for welding. The bottom curb angle might then be welded to the floor-plates.

Another method which might be tried is to rivet an angle-bar along each edge of every plate, as indicated in Fig. 155, the butting angles being easily riveted together from the floor surface when finally assembled in position. At the corners the angles should be mitred, but as the work of preparing them would be almost entirely repetition for all tanks, it need not be costly. Around the circumference of the floor the curb angle should form part of

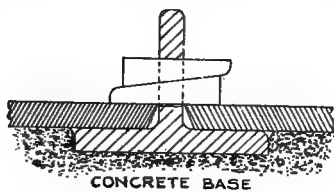


FIG. 154.

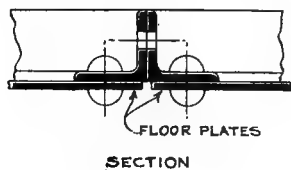


FIG. 155.

the framing for each shaped plate. At points in which transverse joints meet long joints, and also around the circumference, the angles might not come truly together; but the openings could probably be filled with rust cement, or they might be welded.

These suggestions are offered mainly with the object of stimulating interest in the problem, which is obviously worth solving. No matter how difficult a problem of this kind may appear, if practical men can be induced to appreciate the need, and to attack it with interest and determination, a solution is assured.

57. Walls of Cylindrical Tanks.—The walls of cylindrical tanks are invariably built in strakes of sheeting, the circumferential seams being single-riveted lap joints.

The basis on which the sheeting is designed is that referred to in text-books on applied mechanics as the “theory of thin cylinders”; and this is so simple and obvious as to need no detailed discussion here. It is assumed that the liquid pressures, acting radially

outwards, are resisted by direct tension in the ring of sheeting, as indicated in Fig. 156; and this leads to the relation—

$$p d = 2 t f;$$

in which p is the intensity of the liquid pressure in pounds per square inch; d , the tank diameter in inches; t , the plate thickness in inches; and f , the tensile stress in the sheeting in pounds per square inch.

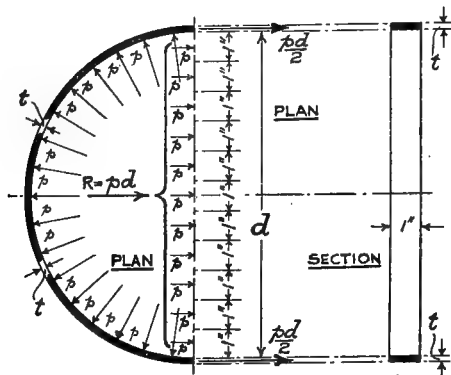


FIG. 156.

The relation may be stated in the form—

$$t = \frac{p d}{2 f};$$

and from this a convenient rule may be obtained for estimating the thicknesses of the various strakes to a first approximation, in preparation for the actual design.

Expressing p as $h \left(\frac{62.5}{1728} \right)$ lb. per sq. in., where h is the liquid head in inches (taking the contained liquid as of weight equal to that of water), and substituting this value for p , the expression may be written—

$$t = \frac{62.5 h d}{2 f \times 1728}.$$

Assuming that one-third of the plate area will be cut away in rivet-holes, t must be increased in the ratio 3 : 2; and if the head

and diameter be expressed in *feet* instead of in inches (their symbols being altered to H and D respectively to denote the change), the expression becomes—

$$t = \frac{62.5 \text{ H D} \times 144 \times 3}{2 f \times 1728 \times 2}.$$

Lastly, taking f as 7.5 tons per sq. in., and expressing the plate thickness in sixteenths of an inch (represented by t_{16})—

$$\begin{aligned} t_{16} &= \frac{62.5 \text{ H D} \times 144 \times 3 \times 16}{2 \times 7.5 \times 2240 \times 1728 \times 2} \\ &= \frac{5 \text{ H D}}{1344} = \frac{\text{H D}}{268.8}; \end{aligned}$$

which, for simplicity in use, may be taken as—

$$t_{16} = \frac{\text{H D}}{270}.$$

As an example, if $\text{H} = 27 \text{ ft.}$, and $\text{D} = 100 \text{ ft.}$ —

$$t_{16} = \frac{27 \times 100}{270} = 10;$$

whence the plate thickness needs to be $\frac{10}{16} = \frac{5}{8} \text{ in.}$

Obviously, the plate thickness necessary will depend upon the diameter and pitch of the rivets; and hence, the thicknesses estimated from this rule will need revision—and perhaps amendment—after the riveting has been designed. It is convenient, however, to have some dependable idea as to the plate thickness when designing the riveting, and for this purpose the rule is likely to be of service.

Now, the diameter is fixed and definite for any given case, but the question arises as to what shall be regarded as the proper liquid head governing the thickness of a strake. The pressure intensity may be taken as varying uniformly from zero at the surface to a maximum at the floor level; and consequently there will be an appreciable difference between the intensities at the top and bottom of a 5-ft. strake, even low down in a tank of considerable depth.

The usual practice is to design each strake for the pressure intensity at its lower edge (except that no strake is permitted to

be less than $\frac{1}{4}$ in. in thickness), but a little consideration will show that this is unnecessarily severe and extravagant:

Consider a strake of sheeting, of height b , subjected to internal pressures as indicated in Fig. 157, the pressure intensity varying uniformly from p_1 at the top to p_2 at the bottom. For uniformity of stress in the sheeting the thickness should vary directly with the pressure intensity (assuming the tank diameter constant); but this is obviously impracticable, and hence the question arises as to what pressure intensity should be taken in estimating the thickness for the complete strake.

Clearly, a thickness based upon the full permissible stress for the pressure intensity p_1 at the top of the strake would be insuf-

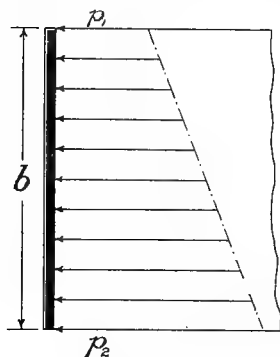


FIG. 157.

ficient at any lower level; while a thickness similarly deduced for the pressure intensity p_2 at the bottom of the strake would be wasteful in that practically none of the material would be working up to its accepted limit of permissible stress.

Here, however, it should be observed that the theory of thin cylinders does not take into account the assistance which must inevitably be rendered to the ring or strake by a diaphragm or end; nor does it allow for the effects of variations in the straining actions

applied to the material by reason of variation in the pressure intensity. The lowermost strake of a cylindrical tank wall is fastened securely to the tank floor by means of the bottom curb; and since this must prevent the material at and near the lower edge of the strake from taking any appreciable strain under load, it follows that this material cannot be severely stressed. As to how far up the strake this restraining influence extends it is difficult to say; and any attempt to form an estimate by analytical methods must of necessity be based upon assumptions which would probably not be realised to an equal degree in any two actual structures. Possibly some information might be obtained from observations with the aid of extensometers applied to the sheeting of actual

anks under all conditions of loading and freedom from load; but even then it is likely that the influences of workmanship and temperature changes—it takes some time to fill or empty a tank of even moderate dimensions—would so obscure the results as to prevent them from yielding reliable evidence with regard to the assistance rendered by the floor to the wall sheeting. But even though no precise and practically acceptable estimate may be possible as to its extent, it is obvious and undeniable that the lowermost strake of the wall sheeting in a tank of the usual and ordinary construction cannot fail to receive assistance from the floor with which it is connected; and the object in drawing attention to the fact here is to adduce qualitative rather than quantitative evidence in support of a suggestion which will be offered presently with a view to effecting economy in the lower strakes of cylindrical tank walls.

From similar reasoning it will be seen that, in the higher strakes, the lower portions of any strake will receive assistance from the upper portions of a thicker strake immediately below. After all, the question is largely one of elastic strains under load; and two pieces riveted together so securely as are the adjoining strakes of a cylindrical tank wall at the circumferential seams cannot take appreciably different strains. Moreover, at the circumferential seams there is a double thickness of material in the lap joints; and this must surely exercise a favourable influence upon the stresses in the upper strakes—though (as will be seen presently) the point is of less practical importance in the upper strakes than in those near the floor.

Let us imagine the strake of Fig. 157 entirely free from restraint as regards horizontal motion due to elastic strain at any level; and let us also ignore any assistance which may be rendered by the upper (and less severely stressed) portions to those below. Then it may be supposed that the strake will be subjected to a tension varying uniformly in intensity from $T_1 (= \frac{1}{2} w h_1 d)$ lb. per inch of depth at the top, to $T_2 (= \frac{1}{2} w h_2 d)$ lb. per inch of depth at the bottom.

Considering a portion of the strake laid out flat, the conditions as regards loading would then be as indicated in Fig. 158, from which it follows that the total tensile force acting upon the strake

will be $\left\{\left(\frac{T_1 + T_2}{2}\right)b\right\}$, while its resultant may be assumed to act in the line T_R , which passes through the centre of gravity of the quadrilateral A B C D in Fig. 158.

Denoting the distance between the resultant T_R and the bottom edge of the strake as kb (where k is, of course, a proper fraction), it is easy to show that $k = \frac{2T_1 + T_2}{3(T_1 + T_2)}$; and if the ratio $T_2 : T_1$ be denoted as r (so that $T_2 = rT_1$)—

$$k = \frac{2 + r}{3(1 + r)}.$$

Now, $T_2 : T_1 :: (\frac{1}{2} w h_2 d) : (\frac{1}{2} w h_1 d)$; whence $r = \left(\frac{h_2}{h_1}\right)$.

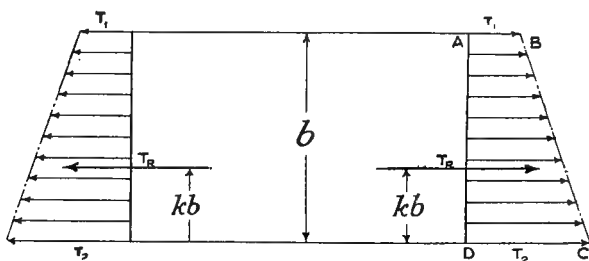


FIG. 158.

Thus, for the lowermost strake of a tank 30 ft. in depth, each strake being 5 ft. in breadth, k will be—

$$\left\{\left(2 + \frac{30}{25}\right) \div 3\left(1 + \frac{30}{25}\right)\right\} = \frac{80}{165} = 0.485.$$

For the next strake above, k will be—

$$\left\{\left(2 + \frac{25}{20}\right) \div 3\left(1 + \frac{25}{20}\right)\right\} = \frac{65}{135} = 0.481.$$

For the next strake above, k will be 0.476; and for the strake immediately below the topmost (*i. e.* the strake for which $r = \frac{10}{5}$), k will be 0.444.

The practical significance of these results may be seen from a numerical example.

Consider the lowermost strake (5 ft. in width) of a tank 100 ft. in diameter and 30 ft. in height, assuming the tank filled to the brim with water or other liquid weighing 62.5 lb. per cub. ft.

The average intensity of tensile loading may be taken as—

$$\frac{T_1 + T_2}{2} = \frac{62.5 \times 27.5 \times 100 \times 12}{2 \times 144} \\ = 7161 \text{ lb. per inch of height.}$$

If the over-all width of the strake be 60 in., only about 55 in. will be subjected to direct tension from its own loading, about $3\frac{3}{4}$ in. being covered by the bottom curb, and $1\frac{1}{4}$ in. at the circumferential seam connecting this strake with that above. Hence, the total tensile force acting upon the plate may be taken as—

$$T_R = \frac{7161}{2240} \times 55 = 176 \text{ tons.}$$

On the basis of the rule for strake thickness deduced in pages 236 and 237, and taking H as 27.5 ft. (*i. e.* above the *middle* of the strake)—

$$t_{16} = \frac{27.5 \times 100}{270} = 10, \text{ giving a thickness of } \frac{5}{8} \text{ in.}$$

The resultant of the tensile forces acting at a height of $0.485 b = 0.485 \times 60 \text{ in.} = 29.1 \text{ in.}$ above the bottom of the strake, the eccentricity of loading will be $30 - 29.1 = 0.9 \text{ in.}$

The effective cross-sectional area of the strake to resist tension may be taken as $\frac{2}{3} \times 60 \times \frac{5}{8} = 25 \text{ sq. in.}$, and the section modulus as—

$$\frac{2}{3} \times \frac{1}{6} \times \frac{5 \times 60 \times 60}{8} = 250 \text{ in.}$$

Hence, the stresses at the extreme fibres would appear to be—

$$f = 176 \left(\frac{1}{25} \pm \frac{0.9}{250} \right) = \frac{176}{25} \pm \frac{158.4}{250} = 7.04 \pm 0.63;$$

i. e. 6.41 tons per sq. in. at the top; and 7.67 tons per sq. in. at the bottom of the strake.

The foregoing discussion and calculation take no credit for the assistance which the strake sheeting will receive through being riveted to the bottom curb. If the lower edge of the flat plate

considered above were held against extension by sensibly rigid stops, as indicated in Fig. 159, it might well be claimed that the additional stress calculated on the basis of eccentric loading should be ignored in designing; and a consideration of the conditions under which the lowermost strake of a cylindrical tank wall must act will show this as a very moderate claim.

It is suggested, therefore, that a strake may be designed on the basis of a liquid head measured above a level midway between its upper and lower edges, as regards both plate thickness and riveting. Had the full 30 ft. head been taken for the example considered above, the estimated plate thickness would have been

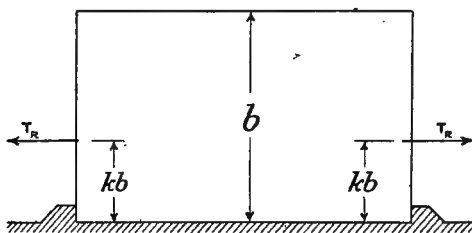


FIG. 159.

at least 10 per cent. in excess of that proposed, showing an appreciable saving for the suggested method.

As regards the upper strakes, it will be seen that the eccentricity of the resultant tension is greater, while the assistance received in restraint is probably less, than for the lowermost strake; and hence the advantage to be gained may be less. But the loading on the upper strakes is much lighter, and the difficulties of designing economically are much less, than for those near the bottom of a tank.

For obvious reasons, no strake should be less than $\frac{1}{4}$ in. in thickness.

The method of designing a strake and its riveting may be shown best by means of a typical example.

Consider the case of the lowermost strake for a tank 100 ft. diameter and 30 ft. in height, as above, the plate thickness having been already estimated provisionally as $\frac{5}{8}$ in.

A rivet $\frac{7}{8}$ in. diameter has a permissible resistance of 5.79 tons

in double shear, and 6.02 tons in bearing in a $\frac{5}{8}$ in. plate. For a total tension of 176 tons, therefore, the net number of rivets required will be $\frac{176}{5.79} = 31$; and as these are to be field rivets, the number should be increased to about 40 in view of the serious consequences which might arise through a defect in one of these seams.

With a pitch of $3 \times \frac{7}{8}$ in. = $2\frac{5}{8}$ in., there is accommodation for, say, $57 \text{ in.} \div 2\frac{5}{8} \text{ in.} = 21$ rivets in a single row; and hence, a double-riveted butt joint, with double covers $\frac{1}{2}$ in. in thickness, will be both suitable and economical.

On the provisionally estimated plate thickness, the net area for resistance to tension will be $\frac{5}{8} \times \{60 - (22 \times \frac{7}{8})\} = 25.47$ sq. in., giving a direct stress of $\frac{176}{25.47} = 6.9$ tons per sq. in., which is satisfactory.

Again, consider the lowermost (5 ft.) strake for a tank 150 ft. in diameter and 35 ft. in height.

The provisional estimate for the plate thickness will be—

$$t_{16} = \frac{32.5 \times 150}{270} = 18,$$

giving a plate $1\frac{1}{8}$ in. in thickness. The total tension in the ring may be taken as—

$$T_r = \frac{62.5 \times 32.5 \times 150 \times 55}{12 \times 2240 \times 2} = 312 \text{ tons.}$$

The net number of $\frac{7}{8}$ in. rivets required would be $\frac{312}{5.79} = 54$; and hence, a double-covered butt joint with three rows of rivets (giving about 64 rivets as against the net 54 required) on each side of the butt would be suitable.

The net area of plate section at the first row of rivets past the edges of the butt covers will be $\frac{9}{8} \times \{60 - (22 \times \frac{7}{8})\} = 45.8$ sq. in., giving a direct stress of $\frac{312}{45.8} = 6.8$ tons per sq. in., which is satisfactory.

From the foregoing examples it will be seen that, with large

tanks, the riveting for the lowermost strakes becomes difficult owing to the large number of rivets for which accommodation must be found. Sometimes the diameter of the rivets is increased in order to reduce their number; and this, within reasonable limits, is good practice, since the shearing resistance of a rivet varies as the square of its diameter. Rivets more than 1 in. in diameter are apt to be very troublesome to close properly, however, especially out in the field; and it is by no means certain that any real saving is effected by their use.

With regard to the suggestion sometimes put forward that the vertical seams should be lap joints, it will be seen that such a course would be the reverse of advantageous in practice. Since the

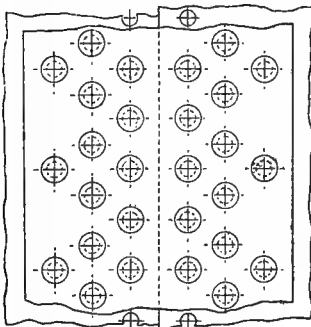


FIG. 160.

resistance of each rivet would be practically halved by reason of its being placed in single shear, the total number of rivets in the joint would probably not be less than with the double-covered butt joint; while the slight saving through the omission of the butt covers would almost certainly be more than cancelled by the necessity for thinning and setting the plate corners—an extremely troublesome matter with such heavy plates and very long laps, and likely

to cause trouble with such high pressures.

Cases may arise in which a saving in the plate thickness of a strake is obtainable by arranging the rivets on some such lines as those indicated in Fig. 160. The increased pitch at the first row of rivets past the edges of the butt covers causes less reduction in the plate section, and thus may render permissible the use of a plate thickness which might otherwise be difficult to justify. It will be clear, however, that the calculated increase in the plate resistance obtainable by such means cannot be very great; while the distribution of loading among the rivets, as well as over the plate section, is highly problematical. Such methods should be used very sparingly; and it is an unfortunate fact that the only cases in which they might be of real assistance are just

those in which the need for reliability is greatest, and the consequences of some relatively slight deficiency likely to be most serious.

The use of a pitch much less than three times the rivet diameter is undesirable not only because it involves a large reduction of the

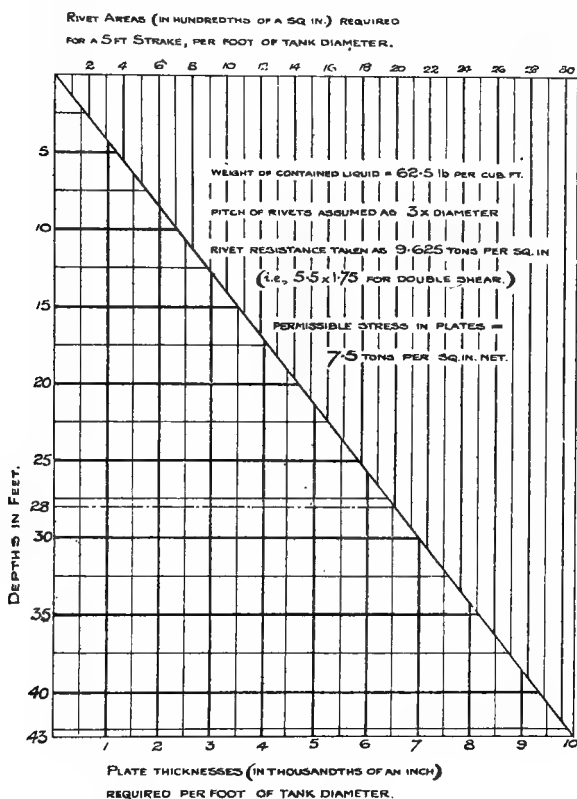


FIG. 161.

plate resistance (thus necessitating the use of thicker plates), but also because it tends to increase the difficulties of riveting by reducing the clearance spaces for the tools.

Trouble may be saved in designing by the construction of a diagram such as that shown in Fig. 161.

The principle on which the diagram is constructed will be clear from the following notes. Since

$$t_{16} = \frac{H D}{270}; t_{1000} = \frac{H D \times 1000}{270 \times 16} = H D \left(\frac{25}{108} \right),$$

$$\therefore t_{1000} = H \left(\frac{25}{108} \right) \text{ per foot of tank diameter. Where}$$

$$t_{1000} = 10, H = \frac{108 \times 10}{25} = 43 \text{ ft. nearly. Total tension in strake}$$

$$= \left(\frac{62.5 H D \times 5}{2 \times 2240} \right) \text{ tons.}$$

$$\text{Rivet area required} = A = H D \left(\frac{62.5 \times 5}{2 \times 2240 \times 9.625} \right) \text{ sq. in.}$$

$$\therefore A_{100} = H \left(\frac{125 \times 100}{17,248} \right)$$

hundredths of a sq. in. per foot of tank diameter. Where

$$A_{100} = 20, H = \frac{20 \times 17,248}{125 \times 100} = 28 \text{ ft. nearly.}$$

Modifications, refinements and approximations will doubtless suggest themselves, both for simplifying the construction of the diagram and for increasing its usefulness.

Since the rivet areas required have been estimated on the basis of shearing resistance in double shear, riveting provisionally designed from Fig. 161 requires investigation with regard to bearing resistance; while appropriate increase in the rivet areas must be made if the rivets are to act in single shear.

CHAPTER VIII

ELEVATED CYLINDRICAL TANKS

58. *Elevated Cylindrical Tanks.*—An elevated cylindrical tank, supported upon a substructure of braced steelwork, may have either a flat floor or a dished bottom, as indicated in Fig. 162.

The flat floor needs a system of deck beams to support it, and this fact is often put forward as a disadvantage of the flat floor

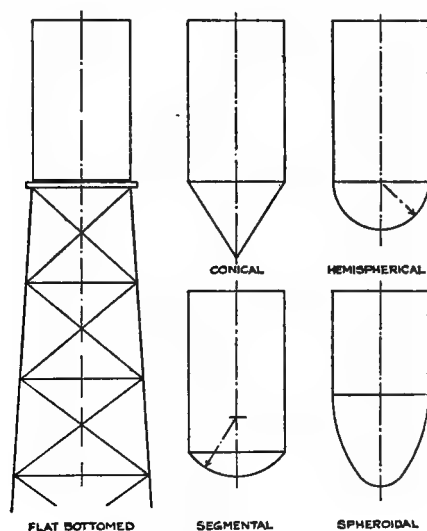


FIG. 162.

by those who favour the dished bottom. It will be seen, however, that with the flat floor the problems of design are simple, the type of work involved in manufacture comparatively straightforward and cheap, and the fabricated material free from special difficulties as regards transport and erection—all factors tending towards economical and rapid production of the structure as a whole.

The dished bottom adds to the capacity of a tank, and may be considerably lighter than the flat floor with its deck beams; but its action is likely to be somewhat uncertain with changes in the head of the contained liquid, while the work involved is obviously expensive, both in manufacture and erection. Besides the troublesome and costly work of bending and riveting the plates, two rather serious practical difficulties are introduced by the dished bottom. First, the connection of the bottom with the cylindrical wall of the tank proper is awkward, and requires great care in design and fitting to ensure adequate strength and tightness against leakage; and second, the support of the tank as a whole becomes complicated through the interference of the suspended bottom.

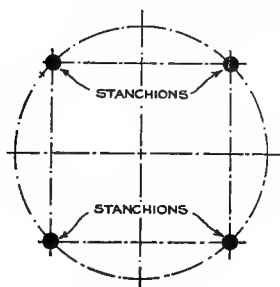


FIG. 163.

59. *Flat-Floored Tanks.*—The roofs and walls of elevated cylindrical tanks with flat floors may be of design and construction similar to those for tanks which stand upon a flat base at or near ground level.

The floor-plates may be arranged on a lay-out similar to those suitable for ordinary tanks, as already described, but the plates must be of such thickness as will permit of their spanning between the supporting joists without exceeding the permissible limits of stress. The methods of treatment from this point of view are, however, not materially different from those already described in connection with the flat floors of elevated rectangular tanks; and hence, no further discussion is necessary beyond pointing out that the joists supporting the floor plates should be arranged to lie at right angles to the main (long) seams, and the transverse seams placed where they will not be subjected to stresses likely to cause them to open sufficiently to become leaky.

Elevated tanks are seldom more than 40 ft. (usually they are in the neighbourhood of 20 ft.) in diameter; and hence the author is of opinion that only four stanchions are necessary for their support, arranged at the corners of a square, as indicated in Fig. 163. By this means, the supporting structure may be braced to form a square tower, with great strength and stiffness in resisting the

overturning actions of wind pressures obtainable by simple and economical means.

An arrangement for the floor joists and beams on this basis, suitable for all ordinary cases, is shown in Fig. 164. The tank may overhang the outermost joists by a distance equal to $0.75S$, where S is the spacing (centre to centre) of the joists in the body of the floor. In these parts, the cylindrical wall will act as a girder in assisting to support the floor and its loading.

The tank should be adequately anchored to the supporting structure to prevent movement and overturning under the action

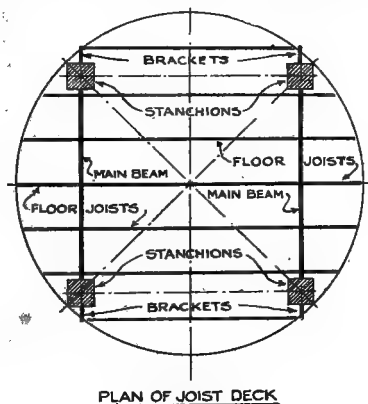


FIG. 164.

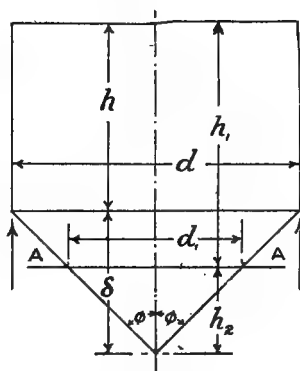


FIG. 165.

of wind pressures. There is little danger of such movement or overturning while the tank is full, or even partly full; but when the tank is empty it may not have sufficient stability by reason of its own mass to resist a sudden and violent wind storm, and the anchorage should therefore be designed on the basis of the empty tank, with a reasonable margin to provide for dynamic effects.

60. *Conical Bottoms.*—It is a comparatively simple matter to deduce mathematical expressions for the stresses in a dished bottom of any given shape—conical, hemispherical, segmental or spheroidal—if suitable conditions be assumed and disturbing influences ignored; but it is open to question as to whether the stresses in an actual dished bottom, manufactured under ordinary commercial conditions, and loaded as such structures must be in real life, bear

a reasonably close relation to those estimated on the basis of any particular set of assumptions.

Consider, for instance, the conical bottomed tank indicated in Fig. 165, containing water (or other liquid of equal density) to the depth shown.

Stresses due to Suspension.—Taking a horizontal section, such as AA, and imagining the lower portion of the cone (with its loading) detached from the rest, the conditions would be as indicated in Fig. 166.

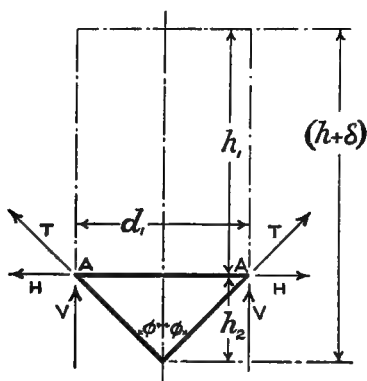


FIG. 166.

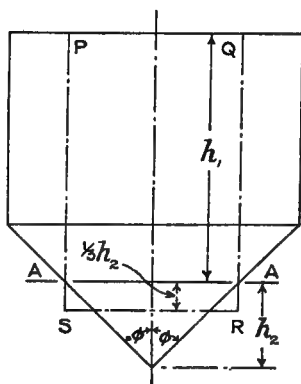


FIG. 167.

The weight of the contained liquid will be—

$$W = \frac{\pi}{4} d_1^2 \left(h_1 + \frac{h_2}{3} \right) w = \frac{\pi d_1^2 w}{12} (3 h_1 + h_2);$$

and this may be regarded as the weight to be supported, for the weight of the sheeting itself will usually be insignificant in comparison with that of the contained liquid—at least when the tank is fairly full.

As the entire weight must be supported around the rim AA, where the circumference is πd_1 , the vertical force (in pounds per inch of circumference) will be—

$$V = W \div \pi d_1 = \frac{\pi d_1^2 w}{12} (3 h_1 + h_2) \div \pi d_1 = \frac{d_1 w}{12} (3 h_1 + h_2)$$

all linear dimensions being in inches, and w in pounds per cub. in.

For water, this becomes—

$$V = d_1 (3 h_1 + h_2) \frac{62.4}{1728 \times 12} = 0.003 d_1 (3 h_1 + h_2) \text{ lb. per inch.}$$

Resolving V horizontally and tangentially, as in Fig. 166—

$$H = V. \tan \phi = 0.003 d_1 (3 h_1 + h_2) \tan \phi \text{ lb. per inch.}$$

$$T = V. \sec \phi = 0.003 d_1 (3 h_1 + h_2) \sec \phi \text{ lb. per inch.}$$

Now, T must be resisted as a tension in the conical sheeting, in straight lines radiating from the apex; and hence, the required thickness of the sheeting, for any given stress, may be readily estimated.

That T will be a maximum at the junction of the cone with the tank proper may be easily seen without calculation; for T is proportional to $\left\{ d_1 \left(h_1 + \frac{h_2}{3} \right) \right\}$, *i. e.* to the area of the rectangle $P Q R S$ in Fig. 167—which obviously reaches its maximum at the upper extremity of the cone.

As an example, in a conical bottomed tank, the cylindrical portion being 20 ft. diameter and 20 ft. in height, and the drop of the cone 10 ft., the magnitude of the tension will be—

$$T = 0.003 \times 240 (720 + 120) 1.414 = 0.003 \times 240 \times 840 \times 1.414 \\ = 855 \text{ lb. per inch of circumference.}$$

At a permissible stress of 7.5 tons per sq. in. (*i. e.* 16800 lb. per sq. in.), and reckoning upon a loss of one-third of the plate section in rivet holes, the required plate thickness (to resist this tension only) would be—

$$t = \frac{855}{16800} \times \frac{3}{2} = 0.08 \text{ (or, practically, } \frac{3}{32} \text{) inch.}$$

The horizontal component (H) of the supporting forces has yet to be dealt with and provided for. It is sometimes stated that the resistance to this component, acting with the resistance to the tension T to produce the necessary vertical reaction, inevitably causes an inward pull to act upon the connection between the cone and the tank proper; but a little consideration will show that this is not necessarily so.

Obviously, the horizontal component of T must be resisted,

and if the cone were supported by a series of separate strips, as indicated in Fig. 168, the connections of those strips would unquestionably be subjected to the action of a horizontal inward

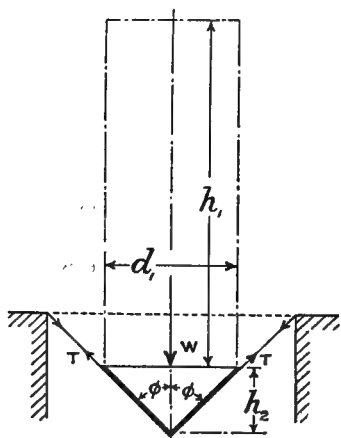


FIG. 168.

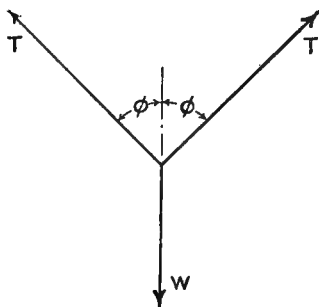


FIG. 169.

(radial) pull all round the circumference. This aspect of the matter is, however, arrived at on the basis of simple suspension only, and the conditions would be exactly the same if the downward load

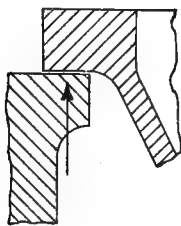


FIG. 170.

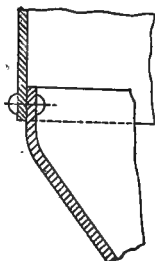


FIG. 171.

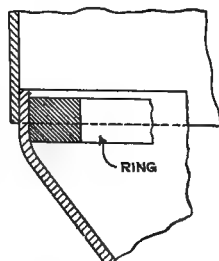


FIG. 172.

were due to the weight of a solid body instead of liquid, standing upon (or suspended from) the junction of two flexible cords, as in Fig. 169.

If the cone were rigid, and supported as shown in Fig. 170, the loading applied to the supporting substructure would be vertically

downward; and this fact provides a simple means for preventing the troublesome inward pull which would otherwise act upon the connection. To simply flange the thin sheeting of the cone, as indicated in Fig. 171, would not be sufficient, for the tendency of the tensions would be to straighten out the flanging to the conical form, and the inward pull would then act as before; but if a ring were provided at the flanging, as shown in Fig. 172, sufficient to prevent distortion of the flanging, the difficulty would be met. The arrangement of Fig. 172 is intended to illustrate the point in principle only, and not necessarily to indicate the manner in which the joint may be constructed in practice; several other ways—perhaps more convenient, and not less effective—are available for securing the desired results,

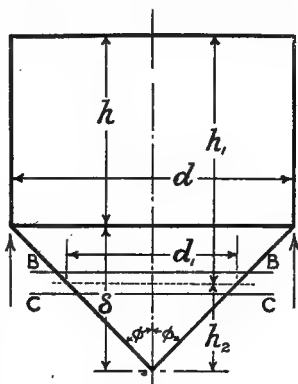


FIG. 173.

and these will doubtless suggest themselves readily.

Hoop Tension.—In addition to the conical tensions caused by the suspension of the bottom, the liquid pressures must be resisted by horizontal hoop tensions in the sheeting.

Consider the ring of sheeting between two horizontal sections BB and CC in Fig. 173. This ring has to support the hollow circular column of liquid which stands upon it; and the conditions as regards

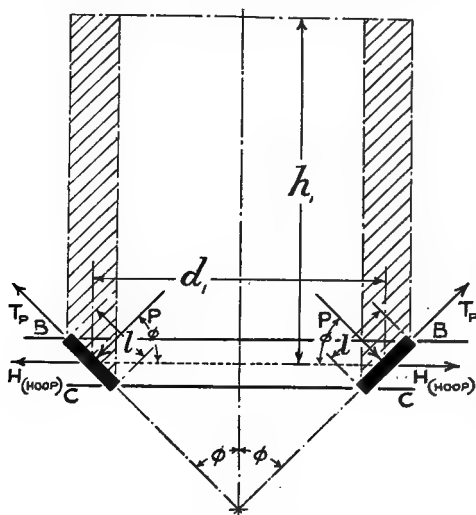


FIG. 174.

loading and support will be seen clearly from Fig. 174, which shows the ring (with the column of liquid which it supports) detached

from the rest. The ring of sheeting is supported solely by the conical tensions T_p as shown, T_p being such part of T (Fig. 166) as is due to the hollow column of liquid standing upon the ring BB CC.

If the inclined depth (BC) of the ring be l inches, the normal pressure P acting upon a strip of it 1 in. in length (perpendicular to the plane of the paper) will be $wh_1 l$. The magnitude of T_p will be $(wh_1 l \tan \phi)$; and the horizontal component H will be $(wh_1 l \sec \phi)$.

This horizontal pressure, acting radially outwards all round the ring, will cause a total bursting force of $(wh_1 d_1 l \sec \phi)$, which must be resisted by tension across two sections of the ring—similar to those shown blacked in Fig. 174—cut by a diametral plane. If the thickness of the sheeting be t inches, and the tensile stress f lb. per sq. in.—

$2 l t f = wh_1 d_1 l \sec \phi$; whence—

$$t = \frac{wh_1 d_1 \sec \phi}{2 f}; \text{ and: } f = \frac{wh_1 d_1 \sec \phi}{2 t}$$

The hoop tension, being proportional to the product $h_1 d_1$, will clearly be a maximum at the top of the cone.

For the example previously considered, and with the same conditions—

$$\begin{aligned} t &= \frac{62.4 \times 240 \times 240 \times 1.414 \times 3}{1728 \times 2 \times 16800 \times 2} \\ &= 0.13 \text{ (or a trifle over } \frac{1}{8} \text{) inch.} \end{aligned}$$

A thickness of $\frac{3}{8}$ in. was found necessary for the conical tension; but in practice, to allow for contingencies, and to provide a reasonable margin for corrosion, it would probably be well to use sheeting $\frac{1}{4}$ in. or $\frac{5}{16}$ in. in thickness.

61. *Hemispherical Bottoms.*—With a hemispherical bottom, the tensile stress, due to suspension, across a section cut by a horizontal plane, may be estimated on a basis similar to that employed for the conical bottom.

Consider the conditions at the section AA in Fig. 175.

The weight of the contained liquid will be (approximately)—

$$W = \frac{\pi}{4} d_1^2 \left(h_1 + \frac{h_2}{2} \right) w = \frac{\pi d_1^2 w}{8} (2 h_1 + h_2).$$

The length of the rim (circumference) at AA being πd_1 , the vertical force to be resisted will be—

$$V = \frac{\pi d_1^2 w}{8} (2 h_1 + h_2) \div \pi d_1 = \frac{d_1 w}{8} (2 h_1 + h_2) \\ = 0.0045 d_1 (2 h_1 + h_2) \text{ lb. per inch of circumference.}$$

Resolving V horizontally and tangentially, as in Fig. 175—

$$H = V \tan \phi = 0.0045 d_1 (2 h_1 + h_2) \tan \phi.$$

$$T = V \sec \phi = 0.0045 d_1 (2 h_1 + h_2) \sec \phi.$$

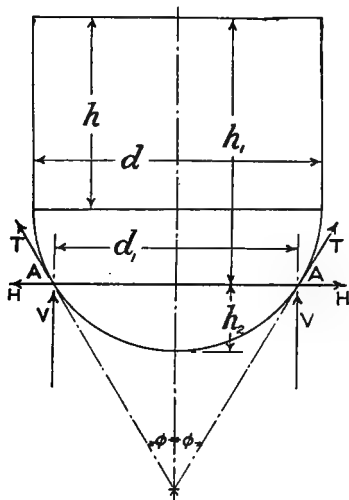


FIG. 175.

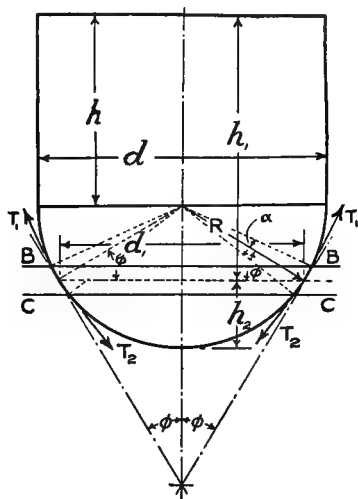


FIG. 176.

In this case, $\sec \phi$ varies from section to section; and near the bottom of the bowl $\sec \phi$ will be very large. Hence it is well to pay particular attention to the lower portions in designing, especially if there be much riveting. Sometimes a stout cap plate is used at the extreme bottom; and in many cases the bowl is partially supported by a central pipe—or “riser”—of considerable strength and stiffness, based upon the foundations.

Hoop Stresses.—Considering a ring of sheeting between two horizontal sections such as BB and CC in Fig. 176, there are two actions causing hoop stresses in the bowl—

will be: $Q_2 = T_2 \sin \frac{\alpha}{2}$. With $\frac{\alpha}{2}$ very small (since the circular measure of a very small angle is sensibly equal to its sine), the radial (inclined) resultant of T_1 and T_2 will be—

$$2 T_1 \sin \frac{\alpha}{2} = T_1 \alpha.$$

Resolving this resultant horizontally and tangentially, the horizontal component will be—

$$H_{(hoop)} = T_1 \alpha \sec \phi.$$

This force acts upon a length of sheeting equal to $BC = R \alpha$; and hence—

$$H_{(hoop)} = \frac{T_1 \sec \phi}{R} \text{ lb. per linear inch.}$$

Acting radially inwards upon the ring all round it, this will cause a compression in the material at each pair of sections cut by a diametral plane.

Thus—

$$2 t f_c = \frac{T_1 d_1 \sec \phi}{R};$$

whence—

$$f_c = \frac{T_1 d_1 \sec \phi}{2 R t} \text{ lb. per sq. in.,}$$

and—

$$t = \frac{T_1 d_1 \sec \phi}{2 R f_c} \text{ inches.}$$

But $T_1 = 0.0045 d_1 (2 h_1 + h_2) \sec \phi$; and substituting this value for T_1 in the expression for f_c —

$$f_c = \frac{0.0045 d_1^2 (2 h_1 + h_2) \sec^2 \phi}{2 R t} \text{ lb. per sq. in.}$$

The net hoop *tension*, therefore, will be—

$$f_t - f_c = \left\{ \frac{0.0361 h_1 d_1 \sec \phi}{2 t} - \frac{0.0045 d_1^2 (2 h_1 + h_2) \sec^2 \phi}{2 R t} \right\} \text{ lb. per sq. in.}$$

For the hemispherical bottom, $R = \frac{1}{2} d$; and $\sec \phi = R : \frac{1}{2} d_1 = d : d_1$. Substituting these values for R and $\sec \phi$, the expression for the net hoop *tensile* stress becomes—

$$f_t - f_c = \frac{9d}{2000t} (2h_1 - h_2).$$

At the junction of the bowl with the tank proper, however, the volume can be expressed more accurately; and hence, *for this level only*—

$$T_1 = 0.003 d (3h + d);$$

giving—

$$f_t - f_c = \frac{3d}{1000t} (3h - d).$$

When the expression for net hoop *tension*, upon evaluation, gives a positive result, the hoop stress is tensile; and when the result is negative, the hoop stress is compressive. An example of the latter condition is illustrated in Fig. 178, the bowl being only partially filled. Above the water level there is no outward normal pressure, and hence the hoop stress is entirely compressive, tending to cause buckling and crumpling of the material—under which action riveted seams in the spherical sheeting are hardly likely to remain tight against leakage. Even for some distance below the surface level, the hoop tensions will not be sufficient to completely nullify the hoop compressions.

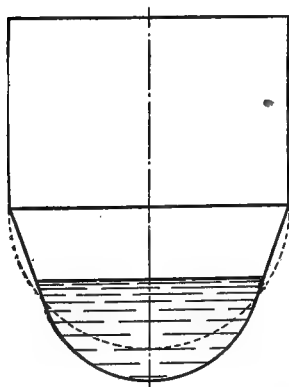


FIG. 178.

The bowl might, of course, be stiffened against inward buckling by means of curved ribs radiating outwards from the extreme bottom, and stayed by horizontal struts or rings—somewhat on the lines suggested for the trough-bottomed rectangular tank in Fig. 113.

62. *Dished Bottoms Generally.*—The segmental bottom, and also the spheroidal bottom may be investigated for stresses on lines exactly similar to those described and illustrated above, and there is, consequently, no need for further discussion of them.

The construction of dished bottoms for tanks—conical, hemispherical and the rest—in this country is a matter of some difficulty, and seems likely to remain so. While the demand is so small, the cost of forms renders the work so expensive as to be practically impossible; and until the cost can be much reduced by the acquisition of stock and standard forms in tank yards generally, the demand cannot well increase. So there is a kind of deadlock.

Moreover, the work of erection is unquestionably of a very troublesome and costly nature, and it is doubtful whether any real saving could be effected in ordinary cases (even if manufacturing costs were reduced considerably) by the adoption of dished bottoms in place of flat bottoms—which latter is the form usually employed for elevated cylindrical tanks in this country.

Under these circumstances, it is neither worth while nor likely to be of practical assistance to give details of construction for dished bottoms; for such details as could be suggested are not in general acceptance, nor have they been used in a sufficient number of cases to warrant a claim to general acceptability.

63. *Substructures and Foundations.*—The design for the stanchions and bracing need not concern us deeply here, for it is a matter of ordinary structural steelwork.

Obviously, the weight of the entire tank and its contents will be uniformly distributed over the four stanchions; and for the horizontal loading due to wind pressures the tower may be regarded as consisting of four framed cantilevers, anchored to the foundations.

At each panel point, the four stanchions should be connected by horizontal diaphragm braces capable of acting either as ties or struts as circumstances may require; but inclined bracings in the diagonal planes should not be necessary in ordinary cases.

It will be clear, upon consideration, that a tower composed of four stanchions is much more simple and straightforward in construction than one composed of six or eight stanchions; and its action is correspondingly more definite, so that its design may be economical without fear of the peculiar and destructive effects which are so often set up, through the influences of manufacture, in structures of a more or less indeterminate character.

In designing the foundations for such structures, it frequently

happens that the most severe conditions obtain when the tank is empty, the structure then possessing comparatively little stability of its own wherewith to resist the overturning actions due to wind pressures.

The necessary stability may be provided by securely fastening the concrete foundation blocks (which must, of course, be of sufficient weight to resist the uplifting tendency, with an adequate margin—usually from 50 to 100 per cent.—to allow for dynamic effects) to the stanchion shafts by means of anchor plates embedded in the foundation blocks and firmly bolted to the stanchion bases.

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